

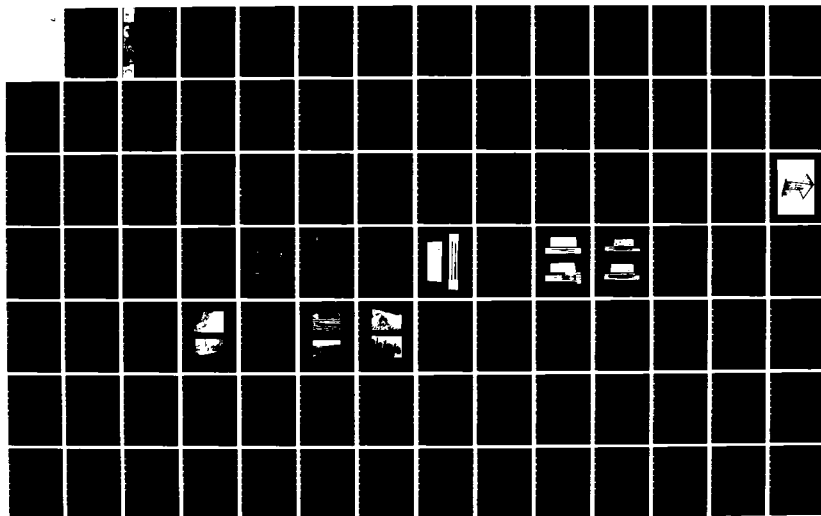
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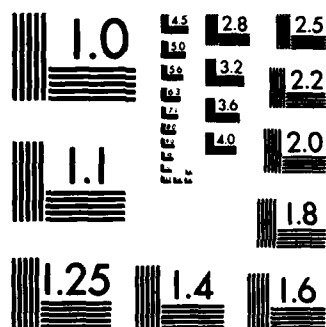
STABILIZATION OF MOVING LAND MASSES BY CAST-IN-PLACE
PILES(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION
VICKSBURG MS GEOTECHNICAL LAB J B PALMERTON MAY 84
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STABILIZATION OF MOVING LAND MASSES BY CAST-IN-PLACE PILES

by

John B. Palmerton

Geotechnical Laboratory

U. S. Army Engineer Waterways Experiment Station
P. O. Box 631, Vicksburg, Miss. 39180



May 1984

Final Report

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20. ABSTRACT (Continued).

mounted on the steel reinforcement bars within the piles) was to provide data so that the resisting mechanism occurring within the pile structure could be determined. At one of the sites (the New York site), the instrumentation revealed data that were consistent with the design assumptions. At the other site (the California site) the results were inconclusive because of post-construction movements and because loads did not occur to any significant extent.

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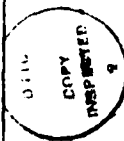
PREFACE

This study was performed under a cooperative agreement between the Federal Highway Administration (FHWA) and the U. S. Army Corps of Engineers Waterways Experiment Station (WES), Vicksburg, Miss. The study was funded under FHWA Order Number 6-3-0067 issued in June 1976.

The work was initiated in January 1976 by the Geotechnical Laboratory (GL) of WES. Project managers during the course of this study were Mr. Z. B. Fry (GL) and Dr. P. F. Hadala (GL). Principal Investigators were Messrs. J. B. Palmerton, Engineering Geology and Rock Mechanics Division, GL, and J. Q. Ehrgott, formerly Soil Dynamics Division, GL. This report was prepared by Mr. Palmerton. Geotechnical Laboratory Chiefs during the period of this study were Mr. J. P. Sale and Dr. W. F. Marcuson III.

Commanders and Directors of WES during the study were COL G. H. Hilt, CE, COL J. L. Cannon, CE, COL N. P. Conover, CE, and COL T. C. Creel, CE. Technical Director was Mr. F. R. Brown.

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CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI)
UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

<u>Multiply</u>	<u>By</u>	<u>To Obtain</u>
feet	0.3048	metres
gallons (U. S. liquid)	3.785412	cubic decimetres
inches	2.54	centimetres
miles (U. S. statute)	1.609347	kilometres
pounds (force)	4.448222	newtons
pounds (force) per square inch	6894.757	pascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
square inches	645.16	square millimetres

STABILIZATION OF MOVING LAND MASSES

BY CAST-IN-PLACE PILES

PART I: INTRODUCTION

Background

1. In 1976, the Federal Highway Administration (FHWA) and the U. S. Army Engineer Waterways Experiment Station (WES) entered into a cooperative agreement to study improved earth reinforcement techniques for use in stabilizing earth masses in place. The use of small-diameter cast-in-place piles to stabilize moving land masses is of particular interest.

2. This report describes the results of two instrumented field projects and presents some of the current concepts utilized in the design of landslide repair by cast-in-place piles.

3. In his classic paper, Terzaghi¹ notes:

If drainage is difficult or its success doubtful, the ground movements can be stopped either by reducing the slope angle or by constructing artificial barriers such as heavy retaining walls or rows of piles across the path of the moving material.

4. One stabilization system, which was patented in the United States in 1969 by Jacques Seidenberg, is the "Reticulated* Pali Radice" (Root Piles) system. This pile system, as it is applied in slope stabilization by Fondedile S.A. of Naples, Italy, and Warren-Fondedile, Inc., of Cambridge, Mass, consists of a system of interlaced closely spaced upslope and downslope battered piles and a pile cap. The piles consist of drilled holes of 5 to 6 in.** in diameter in which a steel reinforcing bar is approximately centered before mortar is tremied into the hole. At intervals, a blast of compressed air is applied to the hole to "seat" the grout surface. Figure 1 is an example of such an installation taken from Bares.² Figure 2 is a photograph of a small-scale model showing the complex alignment of the piles. While this particular system is patented, the concept of the use

* Reticulated means resembling a network or net of the threads of a fabric.

** A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 3.

of piles in other configurations to stabilize landslides has been known to the profession for some time, as is indicated by the quotation given above.

History

5. The use of root piles to underpin building foundations preceded the use of reticulated root piles in control of slopes (Bares²) and underpinning will not be treated here. Several examples of the use of root pile systems to control moving land masses on side hill slopes are documented. Bares,² Cerciello,³ and Fondedile⁴ describe applications of Reticulated Pali Radice in the 1970-75 period to control moving slopes. Figures 3-6 are examples of installation geometries from Fondedile.⁴ According to Engineering News Record,⁵ the installation shown in Figure 3 was the first United States application of the reticulated root pile system to control landslides. The available public information on the system prior to this study provided little understanding as to the design of the structure (i.e., the method for determining the number, depth, size, and spacing of the piles) and the way in which the pile system actually functioned. What was known is the following: (a) the users were generally satisfied after installation because subsequent land movements either ceased or slowed down significantly, and (b) the system was relatively easy to install in congested areas and offered an approach to landslide correction where the real estate necessary for flattening or berming slopes or removing a portion of the driving mass was either not available or was very expensive.

Purpose and Scope

6. The principal purposes of the present investigation were to: (a) develop a method of measuring the longitudinal strains in the steel reinforcing bars in root piles, (b) install instrumented reinforcing bars on two different projects where cast-in-place pile systems were being used to control landslides, (c) collect data from these installations, and (d) try to deduce which of the possible modes by which the structure may resist land mass movement are consistent with the strains in the reinforcing bars.

7. The scope of this report is limited to the design and installation of the longitudinal strain measurement system and the collection and interpretation of strain data from cast-in-place pile installations. One of these installations was on Forest Road No. GLE-7 in the Mendicino National Forest, Calif. The other was on State Route 23A near Palenville, N.Y. These projects were selected because of the interest and cooperative spirit of the owners and their availability during the time period of the WES study.

8. The FHWA-sponsored project at WES covered the period 12 January 1976 - 31 December 1980. Major events in that time period were as follows:

- a. January - March 1976. Planning for instrumentation of a Reticulated Pali Radice installation on U. S. Highway 61 south of Vicksburg, Miss. Work ceased when the State of Mississippi decided on another type of remedial measure.
- b. January - June 1976. Development and field trial of bonded strain gages on reinforcing bars on individual piles at the Gallatin Street project (see Figure 3) in Jackson, Miss.
- c. April 1976. FHWA and WES representatives observed a number of Fondedile S.A. reticulated root pile installations in Italy.
- d. March 1977. At the request of the FHWA staff, resources from this study were used to support field direct shear tests on stone columns on the Steele Bayou Bridge Foundation.
- e. March 1977. Planning and procurement were initiated to support the instrumentation of the reticulated root pile installation on Forest Road No. GLE-7, Mendicino National Forest, Calif.
- f. March 1977. Contract No. DACW39-77-C-0015 let to Fondedile S.A. by WES for a report on reticulated root pile design procedures. This report is available in WES files.
- g. July 1977. Fondedile S.A. began installation on their design on Forest Road GLE-7.
- h. August 1977. Preparation of strain-gaged reinforcing bars for State Route 23A, Palenville, N.Y., begun by WES.
- i. September 1977. Report on Contract No. DACW39-77-C-0015 received.⁴
- j. September - November 1977. The instrumented reinforcing bars supplied by WES were installed at the California site.

Construction was completed in November 1977 by Fondedile S.A. under the supervision of Region 8 of the FHWA.

- k. September - December 1977. Installation of a nonreticulated cast-in-place pile structure by ICOS Corporation on State Route 23A and accomplished under the supervision of the New York State Department of Transportation.
- l. September 1978. Last strain gage readings taken at Forest Road, GLE-7.
- m. April 1979. Last strain gage readings taken at Route 23A site.

PART II: DESIGN OF PILES FOR LANDSLIDE CONTROL

Design Considerations

9. As shown in Figure 7a, stopping a slowly moving landslide without altering slope surface geometry is fundamentally a matter of increasing the available shear force, T , on the failure surface or adding a net force, ΔE , to the lateral earth pressure in the overburden on the downhill side by means of some type of retaining structure. The force T is the product of the shear strength of the soil or rock in the failure zone or plane and the area of the surface. It can be enlarged by increasing the available shear strength of the soil. This is sometimes accomplished by reducing pore water pressure in the failure zone via various drainage schemes. Alternatively, the force T can be supplemented by placing piles across the failure surface, which act as dowels to transmit additional shear forces across the failure plane as shown in Figure 7b. The load capacity of these dowels, ΔT , is limited by the number of dowels and either the shear capacity of the cross section of the root pile or by the ultimate bearing pressure of the soil and the pile's area projection in the vertical plane. The historical use of piles mentioned in the quotation from Terzaghi's paper includes this type of application. While the optimum orientation for such piles is normal to the failure plane, vertical or battered piles or groups of battered piles, such as those in the reticulated root pile system, could possibly function in this manner.

10. Another mode in which a system consisting of uphill and downhill battered piles and a cap could operate is shown in Figure 7c. If the downhill row is closely spaced and the top cap and base remain relatively fixed, they collect load in much the same way a membrane does. The possible role of the upstream pile in this concept is to function as a tension member to hold the cap in place. A third concept of the way in which a reticulated root pile system functions (shown in Figure 7d) and the one espoused by Bares² and Fondedile⁴ is that the piles and cap knit a prism of the overburden material together in such a way as to be the equivalent of a gravity retaining wall capable of transmitting tensile forces as well as greater shear forces than the soil itself is capable of transmitting across horizontal planes. As shown in Figure 2, the reticulated structure is highly complex and three-dimensional.

11. A fourth way in which a pile and cap system may act is as a structural frame, each of whose "legs" function as a retaining wall (see Figure 7e). In this mode each leg experiences a combination of thrust, shear, and bending moment. The moment capacity results, not from the cross section of each individual pile, but from the spacing between piles ("s" in Figure 7e) and the tensile load capacity of the reinforcing bar. In effect, the soil separating the piles in either leg is considered to be like the web in a wide flange beam. Whether the soil can actually transmit shear in such the same way as the web of a beam does is open to question. However, if the system is nonreticulated (i.e., all piles in a given leg are essentially parallel), this assumed model can be easily analyzed. There does not appear to be conclusive evidence that any one of these four modes of behavior or any combination of them is the actual mode of behavior.

12. Slowly moving landslides or those which move in an episodic manner are the only ones to which a root pile remedy can be applied. They represent situations where the additional force necessary to restore equilibrium is relatively small (i.e., $\Delta T \ll T$ or $\Delta E \ll E$). It may, therefore, be the case that the actual forces on the piles are relatively small. If this should be the case, then the dowel action should prove sufficient to restore equilibrium. Major cost savings could be achieved as the cap, mortar, and steel which form the upper level half of each pile would not be needed. Because (a) the reticulated root pile structure is highly indeterminant, (b) the position of the reinforcing bar in the pile cross section and the alignment of the pile cannot be closely controlled in construction, and (c) there are so many different surfaces of the structure on which normal and shear loadings can be applied, it was recognized at the onset of this investigation that the probability of obtaining a definitive answer was not high; nevertheless, it was felt worthwhile to collect whatever data possible relating to the distribution of loads in the pile system. Because it was expected to behave in its linear range, the steel reinforcing bar appeared to be a logical candidate for instrumentation relating to loads.

Design Procedure

Fondedile⁴ S.A. (1977) report

13. In order to gain some insight into the process by which Fondedile

S.A. designed Reticulated Pali Radice, WES contracted with the firm to prepare a report describing design methodology and examples of potential usage. The report stated that the reticulated pile structure functions something like a reinforced concrete beam. Data to support these claims were not presented or referenced. In one example calculation presented, the reinforcing bars are designed to take all the unbalanced shear on an assumed failure plane (i.e., the piles are functioning as dowels). A little later in the same example, the axial loads in the piles are computed as those necessary to resist the foundation loads on a hypothetical pile supported gravity retaining wall. In two other example designs, the piles are assumed to increase the cohesion available on the plane of failure. In the section of this report dealing with the "procedures, calculations, and thought processes involved to achieve a design," the author discusses (a) estimation of loads to be resisted by the active earth pressure on a wall by the Coulomb wedge method with wall friction angle equal to internal friction angle, and (b) an analysis of a circular sliding surface to determine the shear force necessary to restore the desired factor of safety.

14. The author lays out a procedure for designing the pile structure to resist overturning and keep the resultant of the earth pressure and dead load forces in the middle third of the foundation as shown in Figure 8. An analogy to reinforced concrete design in the design process is that the concept of a transformed section of area, A_{trans} , is used. The moment of inertia of the base of the structure, I_{trans} , is computed by assigning equivalent areas of soil to the concrete and steel based upon the ratios of Young's moduli. Extreme fiber stresses are computed as $P/A_{trans} \pm P_e/I_{trans}$ and are kept compressive in the heel of the "wall" by the proper choice of design parameters. The horizontal component, H, of the resultant force on the base of the "structure" is resisted by the combined shear resistance of the soil plus the shear resistance of the piles acting as dowels. The author recommends the piles be extended into rock if possible and says they should always extend below the zone in which failure is suspected.

* P is the vertical component of the resultant force on the base of the structure. $M = P_e$, where e is the eccentricity of P.

15. This reference is difficult to follow and is sometimes contradictory, but the overall impression one obtains of the rationale behind the sizing of the Reticulated Pali Radice is that the authors believe the structure functions as a gravity retaining wall and that no tensile forces should be present in the piles at the "heel" if the piles are properly sized and spaced. If this concept is correct, it is reasonable to assume that the steel could be replaced by more concrete.

ICOS Corporation design

16. The writer's understanding of the rationale for the ICOS pile structures used on Route 23A was gained from examination of the designer's computations which were provided to WES. In this design, each of the upslope and downslope pile groups are assumed to be walls loaded by earth pressures and capable of resisting bending (see Figure 9b, c, and d). These walls function as the legs of a frame or bent. The reinforced concrete cap acts as the cross piece of the frame. End fixity of each member is assumed. The legs are assumed to be fixed at the depth of the failure surface and at the connection with the cap. Bending moment capacity is required of all three members in order to resist the earth pressure loadings on the legs. In the ICOS pile design, all thrust, moment, and shear resistance required of the leg is assumed to be provided by the steel. The concrete serves only as a means of transferring normal and shear loadings from the soil to the steel. The area of the concrete and the pile spacing are used to compute the moment of inertia of the leg cross section used in the indeterminate structural analysis of the frame. Moment capacity is provided by the spatial separation of the steel bars in each leg. The concrete and soil serve to keep the reinforcing bars separated (so that a large moment arm will exist) and are assumed to transmit shear as the web of a wide flange beam does in relation to the flanges.

Commentary

17. Both design methodologies require an estimate of the shear resistance available in the future on a fairly well defined plane of failure. Because the slide is moving or has moved in the past, the strength is, at least for part of the time, slightly less than that required for equilibrium. However, because of changes in pore water pressure and the changes in strength of soils that occur as a result of continued deformation, there is no guarantee that this much strength will be available for all time. Hence, some substantial reserve is required. In the Fondedile and ICOS retaining

wall examples studied, no explicit introduction of a factor of safety was made to account for incomplete knowledge of present or future shear strength. However, the strength parameters chosen in both cases were such that the idealized problem (before application of the correction) had a factor of safety against sliding substantially less than 1.0. Had this actually been the case, slide movements so rapid and so large that corrective measures would not have been practical would probably have occurred. Additionally, the piles in both analyses are sized based on "working" strength parameters for concrete and/or steel rather than on "yield" strengths and the pile axial loads required are well below the actual capacities. Thus, the designer did in effect provide substantial reserves. Further, if the ICOS design concept is correct and the structure acts as a frame with rigid connections, four moment hinges must form before the structure becomes a mechanism. In the Fondedile concept, the design is based on zero tension in the upslope piles. Since the pile and the soil in which it is embedded below the failure plane form a system with substantial tensile load capacity, another source of conservatism exists.

18. In addition to the physical differences (reticulated versus nonreticulated and grout placed under air pressure versus under gravity only) the two pile systems differ in their developer's design assumptions. The Fondedile reticulated pile design attempts to create a highly redundant system which acts as a gravity retaining wall and in which there is no tension in any of the piles. The ICOS parallel pile concept on the other hand is envisioned as acting as a laterally and vertically loaded frame in which some of the piles may be in a tensile state because of bending moments and/or axial loads in the frame. In the ICOS design the concrete is assumed to contribute no strength. Conceptually, neither approach requires that the steel be centered in the pile. This is fortunate as control of the position of the reinforcing bar in an inclined hole is doubtful to say the least.

PART III: FIELD INVESTIGATIONS

19. Field investigations were undertaken at two sites--the New York site and the California site. The California site was selected for an extensive instrumentation program because the developers of the patented reticulated pile system (Fondedile, S.A.) were already involved in the construction of the repair. The New York site was selected for a modest instrumentation program since it offered an opportunity to gather data on a nonreticulated repair structure. Both projects contained similarities of purpose (landslide repair) and both employed the use of small-diameter cast-in-place piles.

20. At the time both projects were begun, the possible role of the cap beam as the cross piece in a structural frame (Figure 7e) was not recognized. Had it been, alternative forms of instrumentation (i.e., to measure moments in the cap) might have evolved. At the time the project was begun it was decided that the measurement of longitudinal strains on the surface of the steel reinforcing bars was the only measurement that (a) would bear any close relation to the earth pressure loadings being transferred into the pile structure above the failure surface, and (b) had a reasonable chance of being successfully carried out in the field.

Instrumentation

21. Instrumentation used in this program consisted of foil strain gages bonded to specially prepared sections of reinforcing bars.

22. Reinforcing bars used for construction of the piles consisted of No. 10 bars at the New York site and No. 9 bars at the California site. The No. 10 bars are spiraled (coarsely threaded) and come from the manufacturer with mating threaded couplings to permit easy joining of the bars. Ten-foot lengths were used in the field; therefore, these same type and length bars were used for the instrumented piles. The No. 9 bars were common reinforcing steel (grade 60) and were instrumented in 20-ft lengths.

23. After the bars were received by WES, the positions for gage locations were marked off and each bar was lathed down at that location for a length of 6 in. to produce a smooth uniform section. The bars were lathed to diameters of 1-3/16 in. (No. 10 bars) and 1-1/16 in. (No. 9 bars). Figure 10 shows one such bar after lathing.

24. Prior to gaging each set of bars making up a pile, the bars were joined together and the bar orientations were marked. Therefore, all the gages oriented in the north direction along a given pile would line up when rejoined in the field. Punch marks were placed on the bars. Also, color-coded marks were placed at the ends of each bar. The punch marks are visible in the photograph in Figure 10, just below the 2- and 6-in. marks on the scale.

25. Full bridge circuits were used at each gage location rather than individual gages or half bridges. The reason for full bridges was to minimize detrimental influences such as temperature effects, and to maximize gage output to increase reading resolution. Figure 11 shows details of the bridge circuit and the orientation of gages on the bar. Foil strain gages, 1/4-in. long, were used and placed on the prepared bars with a commercially available epoxy commonly used to bond strain gages for long-term tests. The individual strain gage wires were connected to a terminal strip completing the full bridge circuit. Figure 12 shows one such completed circuit.

26. Small, four-wire shielded cable of predetermined length sufficient to reach from the particular gage circuit up the length of pile to a common recording station alongside the road was added. Waterproofing was placed over the gages, wires, terminal strip, and around and under the cable wire ends (Figure 13). This particular waterproofing is not abrasion-resistant, but is fluid enough to fill small voids and to seal the gages and wires.

27. A final waterproofing coat was applied to provide durability and abrasion protection to the gages. This coating is shown in Figure 14. Note that the ends of the cables have been completely covered. The exposed ends at the other end of the cable were also sealed by removable steel tubes sealed with rubber grommets. Moisture is one of the greatest causes of gage malfunction and extreme care must be taken for moisture protection.

28. Finally, wraps of tape were applied to the bar to provide additional abrasion protection (Figure 15). When complete, each circuit was balanced and a zero recording made on an SR-4 indicator. The resistance of the insulation was determined by a megohmmeter. The bars were placed in a water bath for several days and the insulation resistance monitored. If any circuits showed signs of poor insulation or of insulation deterioration, that bar was regaged.

29. Each completed rebar for the California project was pulled in tension with 1000- and 2000-lb load increments to verify the circuit output.

This verification was not done on the rebars for the New York site because of time constraints. However, the instrumentation for the New York project was prepared in the same manner (and by the same personnel) as for the California site. The bars were checked in the field prior to placement by flexing each bar in a known orientation and noting the direction (tension or compression) of gage output on the SR-4 indicator.

Recording

30. For this project, use of a manually operated SR-4 indicator was chosen as the most practical recording system. Figure 16 shows a typical data sheet which was used to record data from each gage circuit. The pertinent data regarding the gage circuit are recorded in the remaining columns--date of reading, time, weather, normal and reverse readings, and remarks. The columns marked "half difference," "indicator zero," and "strain" are calculated from the normal and reverse readings.

31. Two readings were made of each gage circuit--one with the terminals normally connected so that increasing numerical values indicate compression and another with the terminals reversed. The value "half difference" computed from those readings is the offset of the circuit output from the indicator's zero value. The indicator's zero value is computed by adding the half difference to the lower numerical value of either the normal or reverse reading. For the indicator used on these projects, the zero value should be $29,995 \pm 10$. A value outside that range indicates (a) a misreading of one or both of the normal and reverse readings, or (b) a malfunctioning of the indicator, e.g., low batteries.

32. Half difference is used to compute the strains occurring in the bar. Numerical changes in the half difference over a period of time represent changes in gage output due to strain occurring in the steel rebar. Since a full bridge configuration is used with two gages oriented in the axial direction and two gages in the lateral direction, true strain can be calculated by dividing the relative change over a period of time by a factor of: $[(2)(1 + \nu)]$, i.e., 2.5., where ν is Poisson's ratio for steel; assumed to be 0.25.

The California Site

Site description

33. The location map for the site is shown in Figure 17. The site is located approximately 30 miles northwest of Willows, Calif., on Forest Highway 7. The site is within the Mendocino National Forest. The principal use of the highway is to facilitate logging operations. This portion of Forest Highway 7 is relatively new, construction commencing in 1964. Since the end of construction of the highway, slide movements have occurred at numerous locations.

34. The portions of the highway for which the reticulated pile repair system was selected are located between highway stations 272+10 and 275+20. A description of the site geology and the slide activity prepared by the State of California Department of Transportation⁶ is given below.

Site geology

35. The site of the slide under investigation between stations 271+ and 276+ is located near the eastern edge of the Coast Ranges Geomorphic Province of California. This province consists of many separate ranges, coalescing mountain masses, and several major structural valleys.

36. Two entirely different basement complexes, one being the Jurassic-Cretaceous assemblage called the Franciscan formation and the other consisting of Early (?) Cretaceous granitic intrusives and older metamorphic rocks, are present in this province.

37. The area of study has been mapped by the California Division of Mines and Geology as part of the Franciscan formation. Geologic units mapped on site by the California Department of Transportation, however, are not the typical graywacke, shale, chert, and conglomerate of the Franciscan but rather are characterized by metasedimentary rocks consisting predominantly of phyllite with secondary mica-quartz schist and slate. These rocks may be metamorphosed Franciscan rocks, but are more probably part of an older basement complex mapped in adjacent areas as Pre-Cretaceous metasedimentary rocks.

38. Intermediate in metamorphic grade between slate and schist, the weathered surficial outcrops of phyllite display a rich golden silky sheen on the surface of cleavage or schistosity and have a greasy feel when rubbed with the hand. Unweathered phyllitic core samples taken below the water table were dark blue-gray, with all gradations in color from the capillary

fringe to the outcrops on the surface.

39. Interbedded with the phyllite are scattered thin layers of mica-quartz schist which are light to dark gray in color and are much harder than the phyllite. Local pods and veins of quartz are common, along with numerous zones of highly sheared and pulverized phyllite which have taken on the characteristics of clayey gouge.

40. The slide mass itself is composed of a mixture of reddish-brown clayey soil and rock fragments of the parent phyllitic and schistose bedrock.

41. Excellent exposures of the bedrock are visible on either side of the slide in the cut slopes. Here the thinly bedded metasediments display severe folding, faulting, and fragmentation. This distortion is probably related to ancient movement along the major northwest-southeast trending Stoney Creek fault lying approximately 4 miles to the east.

Description of slide

42. Forest Highway 7 crosses the middle of an old landslide in the vicinity of Sta 271+75 to 275+. Examination of aerial photos taken on 10 October 1964 during clearing and grubbing operations for the new alignment shows the main scarp to be located approximately 200 ft right of the center line. The toe of the old slide may extend to Rattlesnake Creek at the base of the steep hillside (Figure 18).

43. Renewed movement occurred during the wet season of 1973. Cracks progressed up the cut slope and into the dense brush for a total distance of approximately 160 ft right of the center line. The main arcuate-shaped scarp is 5 to 6 ft high with some cracking above it.

44. Movement on the downhill side also occurred with subsequent displacements of the roadway and sidehill embankment. Cracking and pushouts of loose material can be observed for a distance of approximately 200 ft downslope (Figure 18).

45. Guardrail for some 100 ft left of the center line has dropped a few feet and is partially suspended in the air. The underdrain which crosses the roadway near station 273+60 was still functional. The outflow of this underdrain was deeply incised into the steep hillside by the erosive action of runoff.

46. It appears that failure was precipitated by saturation of old, loose slide debris forming the 20 to 30 ft high cut slope and the embankment foundation. Removal of lateral support by cutting undoubtedly contributed to the renewed instability.

47. A somewhat flat, amphitheater-shaped area below the main scarp and above the top of cut serves as a catchment area for runoff, contributing to the saturation of the slide debris.

Details of construction

48. The construction work for the subject landslide repair was performed by Warren-Fondedile, Co., Cambridge, Mass., under contract with the FHWA. The major features of the repair work consisted of the construction of the pile cap, the drilling of 721 5-in.-diam boreholes, and the placement of grout and reinforcing bars within the boreholes. Field supervision, layout, and drilling was performed by Italian nationals affiliated with Fondedile S.A. of Naples, Italy.

49. Figures 19-21 are construction drawings showing pertinent details. Figure 19 shows the front view of the wall, Figure 20 is a plan view indicating the numbering system, and Figure 21 is a cross-sectional view of the structure.

50. The first construction item was the 310-ft-long capping beam. The beam was 3 ft thick by 6 ft wide and consisted of steel-reinforced concrete. Plastic sleeves were embedded within the forms before the concrete was poured. These sleeves (see Figure 22) were situated at the planned pile locations and were battered at the required angle. The capping beam also served as a mat on which the drilling equipment could operate.

51. Following the construction of the capping beam, drilling commenced. The uncased borings were 5 in. in diameter and were drilled to depths ranging from 50 to 80 ft. The drill rigs were manufactured by Fondedile, S.A., Naples, Italy (see Figure 23). The borings were sequenced in a more or less random fashion, the criteria being not to drill a new borehole in the vicinity (e.g., 30 ft) of a freshly poured pile. After a boring was completed, a No. 9 (1-1/8-in.-diam) reinforcing bar (grade 60) was placed in the hole. The reinforcing rods were in 20-ft lengths. These lengths were overlapped and welded together as they were lowered into the borehole. A tremie pipe (2-in.-diam.) was then inserted into the borehole (alongside the reinforcing steel) and the grout was pumped into the hole. No attempt was made to insure that the bars were centered in the concrete. Indeed, due to the batter angle and the insertion of the tremie pipe, it would be impossible for the bars to be centered within the borehole.

52. Typically, the boreholes of 50 ft depth could be bored in a period of two to three hours. The placement of the steel and the grout commonly took an additional two hours. The borings were accomplished with a rock bit using compressed air as the drilling fluid. With the exception of a few boreholes drilled to depths in excess of 80 ft, no ground water was encountered.

53. Figure 2 is a photograph of a built-to-scale plastic model. It is evident from the photographs that the piles are tightly interlaced. Near the vicinity of the pile cap, the piles almost touch one another. The pile density is 2-1/2 piles per ft of retaining structure. During the actual construction, it was not uncommon for a new boring to intersect and go through a previously placed pile. This would not necessarily be evident during typical construction. However, when pile 376 was bored, the bit struck instrumented piles 372 and 371. This was made evident due to pieces of strain gage electrical cables being blown out of the new borehole. The electrical cables were attached to the reinforcing bars of piles 372 and 371.

Instrumented piles

54. Figure 24 shows a plan view of the pile layout. This view shows the repetitive nature of the system. The repetition number is 28; that is, the value 28 (or a multiple of 28) must be added to any given pile location number to obtain the location of a pile that is similar (with respect to its neighboring piles) to the given pile. Included in the 28-pile unit:

- a. Four piles are vertical (Type C).
- b. Four piles are battered downhill and upstation (Type A).
- c. Four piles are battered uphill and downstation (Type A').
- d. Four piles are battered downhill and downstation (Type B).
- e. Four piles are battered uphill and upstation (Type B').
- f. Four piles are battered uphill (Types D', E', F', G').
- g. Four piles are battered downhill (Types D, E, F, G).

55. At the onset of the project, it was planned to instrument 18 piles within a repetitive unit. With one exception, this plan was accomplished. The exception was pile 373, a Type A' pile. Because of construction difficulties, it was not possible to install this instrumented pile. The instrumented bars originally planned for pile 373 were later placed in pile 404, a Type B pile. The instrumented piles are indicated by the darkened circles shown in Figure 24. The following pile types were instrumented:

- a. Two Type A piles (piles 377 and 384).
- b. One Type A' pile (pile 380).
- c. Three Type B piles (piles 376, 383, and 404).
- d. Two Type B' piles (piles 374 and 381).
- e. Two Type C piles (piles 375 and 382).
- f. One each Type D (pile 371), D' (365), E (372), E' (378), F (379), F' (385), G (392), and G' (386).

56. A cross-sectional view of the instrumented section is shown in Figure 21. The planned elevations of the gaging stations are shown on this figure. Each instrumented pile had a total of 28 full-bridge foil strain gages. These gages were placed in groups of four (located at 90-deg intervals around the reinforcing bar) at seven depths along the bar. The depths below the top of the pile cap at which the gages were placed were 4, 10, 18, 26, 34, 39, and 43 ft. The instrumented sections were located at approximately station 273 + 60.

57. The instrumented reinforcing bars were fabricated at WES and shipped by truck to the project site. Each instrumented pile installation required three individual No. 9 reinforcing bars. The first, or top, bar contained the gages to be placed at the 4- and 10-ft depths. The second, or middle, bar contained gages for the 18- and 26-ft depths and the third, or lower, bar contained the gages for the 34-, 39-, and 43-ft depths. The maximum depths to which the instrumented bars could extend varied from 48 to 50 ft (depending on the batter angle). In cases where the borehole exceeded the maximum depth, a length of No. 9 reinforcing bar, provided by the contractor, was welded to the lower bar.

58. As soon as the contractor had completed the drilling operations, the instrumented bars (and any necessary extension) were manually lowered into the hole one at a time. The bars were overlapped 2 ft and tack welded together.

59. Figure 25 shows the reinforcing bar centering devices used for the instrumented piles. (The contractor did not use centering devices for other piles.) These centering devices were attached to provide protection for the electrical cables attached to the strain gages. These devices consisted of short (10-in.) strips of 1/8-in.-thick steel bent to stand away from the reinforcing bar. The steel strips were attached by screw-type hose clamps and were placed at approximately 10-ft intervals along the bars in a spiral-like fashion.

60. The ends of the instrumented reinforcing bars were stamped and color-coded so that the orientation of the bar could be visually maintained during insertion and welding. During insertion, the cables extending from the instrumented bars were taped to the bar at 10-ft intervals. Channels emanating from the borehole were provided (Figure 26) within the capping beam. Following the placement of the bars, the borehole was grouted and the channel containing the cables was filled with grout. The instrumented boreholes were grouted from the surface because the tremie would not pass the cables and strain gages without damage to the cables and gages.

61. The electrical cables extending from the cap beam channels were then placed alongside the cap beam and led to the gaging station (Figure 27). The gaging station basically consisted of three 55-gal oil barrels into which the electrical cables were inserted. The cables entered near the bottom of the barrels via short lengths of plastic pipe. As soon as construction ended, backfill was placed over the plastic pipes and part way up the barrels (Figure 28). A plywood structure was placed over the barrels.

Strain data

62. The electrical cables leading from each strain gage were labeled (with a metal tag) to indicate the pile number and position of each strain gage. The label took the form of a number indicating the pile location, followed by a letter giving the depth to the gage, and another number giving the orientation. Thus, a label such as 375 D 3 means pile number 375, the fourth (D) depth down (26 ft), with the gage oriented downslope (3). The orientation numbers are 1 for upslope, 2 for downstation, 3 for downslope, and 4 for upstation. In a few instances, it was not possible to orient the No. 1 mark in an upslope direction; however, any discrepancies were accounted for in the data reduction process.

63. Strain gage readings of instrumented piles were taken immediately after the grout had hardened. Thereafter, recordings were generally made at intervals of one day, three days, one week, and then two week intervals after gage placement. This recording schedule was maintained while WES personnel were present at the project site. After construction was completed, WES personnel returned to the site every few months to make recordings until it was established that the strains had reached equilibrium values.

64. Figures 29-64 are plots showing the strain gage data plotted alongside a profile of the pile. For example, Figures 29 and 30 are composed of the strains recorded on pile 365 for 14 individual gages, those gages located upslope and downslope at the seven gage depths. As seen on Figure 29,

the number 2 indicates those gages situated on the upslope side of the reinforcing bar, and the number 1 those situated downslope. The batter angle ($\theta = 14.6$ deg) is the apparent angle of the pile as seen from a position located downstation from the pile. Figure 30 shows the strain for the 14 gages located up and downstation on pile 365. The number 2 corresponds to upstation and the number 1 to downstation. The batter angle ($\theta = 0$ deg) is the apparent angle of the pile as seen from an upslope position. The strains are given in microstrain, i.e., one microstrain equals 1×10^{-6} . The scale is given on the right side of each plot.

65. With very few exceptions, the strain data as shown in Figures 29-64 indicate that the reinforcing bars within the instrumented section are in a compressive state. The trend of the data shows that following installation the strains built comparatively rapidly for one or two months (the construction season). After that, the strains either remained constant or relaxed slightly. Tension occasionally occurred near the pile cap; however, there was no apparent trend.

66. Following the completion of construction (December 1977) the compressive strains typically ranged from 25-250 microstrains indicating compressive stresses (assuming Young's modulus $E = 30 \times 10^6$ psi) within the reinforcing steel ranging from 750-7500 psi. An average value is approximately 75 microstrains. After the installation of all piles within the project, the area over the 6-ft-wide pile cap was backfilled to a depth of approximately 5 ft; that backfill becoming the shoulder of the reconstructed highway. Since a repetition unit of 28 piles is 12 ft in length, the weight of the backfill (assuming a density of 100 pcf) over the 12-ft length would be 36,000 lb. If it is assumed that this weight is distributed to the 28 piles, this would lead to a maximum compressive force within the piles of approximately 1300 lb. If this stress is all taken by the steel (the cross-sectional area of a steel bar being 0.994 sq in.), then the strain within the steel due to the backfill would be on the order of 43 microstrains. It should not be expected that the backfill load would be evenly distributed among the piles due to variations in batter angle, soil and rock conditions, and inhomogeneities within the pile cap. The situation is statically indeterminate.

67. The following trends in the strain data are considered significant:

a. Immediately following the installation of an instrumented pile, small (on the order of 10-20 microstrains) strains developed. Near the pile cap, these small strains were often tensile.

b. During the construction period, the strains generally (but not always) increased compressively to values typically on the order of 30-80 microstrains. Near the pile cap, these strains were often tensile.

c. After the placement of the backfill (December 1977) the strains typically increased compressively rather rapidly to values ranging from 25-250 microstrains. For those strain gages that continued to function following the end of construction, a relaxation of strain generally occurred (several months later).

Slide repair performance

68. The rainfall data at Black Butte Reservoir are shown in Figure 65. Black Butte Reservoir is located some 20 miles northeast of the project site. Therefore, these rainfall data should roughly apply to the project site. The rainfall data indicate that the rainfall during the winter of 1978 (just following construction) was quite above the mean of the previous two years. The completed pile structure performed well during the rainy season. No cracks or movements were noted within the reconstructed roadway. Large strains, or strain buildup, within the instrumented section did not develop through the time of the last instrument readings (September 1978).

The New York Site

69. In early 1977, the WES was advised of a landslide in the state of New York (NYS) which required the stabilization of a road located in Kaaterskill Clove in the Catskills. The road (Route 23A) was located on the side of a mountain serving as a major route connecting the Hunter Mountain resort area and Haines Falls, to Palenville and Catskill, New York. Any corrective action required that one lane of the road be kept open during construction. This meant construction had to take place within the width of one lane adjacent to a sharp drop-off. The New York State Department of Transportation* (NYSDOT) during the design phase "believed that the only feasible way to stabilize the area with the constraints issued was the patented procedure developed by the Fondedile Corporation." For this

* Letter from New York State Department of Transportation, Soil Mechanics Bureau, to Construction and Maintenance Division, FHWA-USDOT, subject: PIN1124.03(01), Contract No. D95475, Route 23A, Kaaterskill Clove, Green County, dated 26 August 1977.

reason, specifications were prepared requiring that an earth-pile retaining wall be "designed and constructed in accordance with the Fondedile patent or be an approved equal."

70. The low bidder for the project was I&OA Slutzky, which had submitted a design prepared by ICOS Corporation of America. The design was somewhat similar to the Fondedile* technique in that it involved small cast-in-place piles with a single reinforcing bar in the pile center. The system was different, however, in several major features. The pile grouping was not a complex network of interfingering (reticulated) piles but rather a two-dimensional layout with the piles battered at 15 deg upslope and downslope. The design did not require that the piles be anchored into bedrock. Also, unlike the Fondedile system as constructed in California, the piles were placed prior to construction of the cap beam. NYSDOT considered the ICOS design an approved equal. Two features of the NYS project were attractive as a study case for WES. First, portions of the landslide at the project site were active at the time of construction and were expected to produce loadings on the pile system. (Although the California slide was intended to be the major WES study site for this program, the slide was recognized during installation to be dormant, and it was uncertain whether reactivation would occur.) Second, the ICOS design, being two-dimensional, was simpler to visualize and analyze.

71. Because of the differences of the system design and the possibility of the landslide producing large pile loadings, WES expanded its study to provide limited instrumentation at the New York State project. This was possible since New York State agreed to provide operational support in the field and to provide necessary personnel to make long-term gage readings after the piles had been placed.

72. The plan called for instrumenting bonded strain gages to seven piles of the system. Each pile would be instrumented at two or three depths. Five of the piles had gages oriented 180 deg apart in the upslope and downslope directions. Two piles had gages oriented at 90 deg intervals. The rebars were prepared and gaged at WES during the period from 24 October 1977 to 4 November 1977. Subsequent readings of the gages were made by New York State personnel.

* Letter from Warren-Fondedile to New York State Department of Transportation, Soil Mechanics Bureau, subject: P.I.N. 1124.05-010, Route 23A Haines Falls-Palenville, Green County, Proposed Special Specifications, dated 8 August 1975.

Site description and history

73. A location map of the site is shown in Figure 66. The site is located within the Catskill State Park approximately 1-1/2 miles west of Palenville, N.Y., on Route 23A. The road is a main route between Palenville and the Haines Falls-Hunter Mountain area.

74. A plan view of the road area in the vicinity of the project (centered about station 50+00) is shown in Figure 67. This portion of the road was constructed on the side of a mountain utilizing rock retaining walls and timber cribs keyed into the underlying bedrock.

75. In June 1971, the NYSDOT made an inspection of the road between Palenville and Haines Falls and in February 1972 recommended treatment of the deteriorating roadway within the subject area. A slope indicator was installed at station 50+00 in November 1975. The readings⁷ indicated slope movement within the upper 20 ft of the roadway. The rate of movement was interpreted as 0.07 in. per month (0.8 in. per year) at a 10-ft depth and 0.05 in. per month (0.6 in. per year) at a depth of 19 ft. In addition, the existing rock wall on the downslope side of the road moved outward at rates of movement of 0.2 to 0.4 in. per month.

76. The general cause of movement was attributed to oversteepened slopes, inadequate drainage, and deteriorating timber cribs. A contract was let (NYSDOT Contract RCR 75-59, March 1977) which provided for the construction of a Sta-Wal in the vicinity of station 50+00. The purpose of the Sta-Wal was to provide temporary stabilization. This construction was accomplished by excavating the outside (downhill) road lane down to the base of the old rock wall, placing the Sta-Wal units, and backfilling with gravel. Photographs taken by NYSDOT along the road at the project are shown in Figure 68. Movements continued to occur within the Sta-Wal and the underlying timber cribs in the vicinity of station 50+00. The Sta-Wal construction extended from stations 49+64 to 50+25.

77. The project site is 0.14 miles long from NYSDOT stations 45+69 to 53+13 and is situated approximately 100 ft above the flowline of Kaaterskill Creek. The average downhill slope from the edge of the roadway (at station 50+00) to the south (near) bank of the creek is approximately 1.3 vertical to 1.0 horizontal or slightly in excess of 45 deg. A contour map of the soil surface is shown in Figure 69. The bedrock contour map is shown in Figure 70. Borings obtained by NYSDOT at station 50+00 describe the subsurface as being composed entirely of a moist, very compact, glacial till containing boulders down to a depth of approximately 60 ft (from the

roadway surface) where bedrock consisting of a green and red shale was encountered. No evidence of slippage planes was noted on the driller's logs nor were soil samples obtained. The profile of the roadway down to the creek is shown in Figure 71. This figure also indicates the location of the new pile construction and other pertinent details.

Design of pile structure

78. The ICOS* design was chosen for installation. Essentially, the pile arrangement consisted of a two-dimensional pattern of battered (upslope and downslope) 4-in.-minimum-diam cast-in-place piles, each pile containing a single No. 10 (1-1/4-in.-diam) reinforcement bar. A reinforced concrete pile cap, placed after the casting of the piles, completed the system.

79. Five conditions were assumed during the design process. The considerations pertinent to three of the designs are shown in Figure 72. The design for types X and Y yield a four-pile arrangement (two piles battered upslope and two piles battered downslope). The type Z arrangement yielded a five-pile arrangement, the additional pile battered downslope. (The other two design conditions were for a culvert crossing and for a condition similar to type Z, but containing a wider pile cap.) Type X is characterized by a single shear plane, type Y by a double shear plane.

80. The type Z design was used in the vicinity of station 50+00, the location of this report's study area. A brief description of the design approach is illustrated in Figure 73. The general approach was to design a space frame retaining structure to resist sliding of the soil above the assumed shear plane. The soil properties required for the analyses were the angle of internal friction (assumed to be 30 deg) and the soil density (assumed to be 130 pcf). Pile strengths were calculated assuming a working diameter of 4 in., a single No. 10 grade 60 reinforcing bar, and a grout strength of 4000 psi (28-day strength). Figure 73a schematically shows the location of the assumed shear plane in relation to the battered piles and the cap. The upslope pile cluster resisted the soil on the upslope side of the piles. The downslope cluster resisted the soil in between the up and downslope clusters.

81. A vector analysis was used to derive the forces on each pile cluster (Figure 73b).

* Letter from George Tamaro, ICOS Corporation of America, to John Ehr Gott, U. S. Army Engineer Waterways Experiment Station, with copies of analysis and design computations, dated 4 November 1977.

82. The properties and stiffness factors of the pile cluster and pile cap were calculated using the assumed minimum dimensions of the piles (i.e., 4-in. diameter with 24- or 18-in. spacings) and cap. A triangular transverse load distribution was assumed to act on each pile cluster. The pile clusters were idealized as beams with fixed ends (Figure 73c).

83. Moment distributions were calculated without and then with a sidesway correction using the Hardy Cross method. The combined loading diagram is shown in Figure 73c.

84. Once the loadings on the pile clusters were known, the individual piles within each cluster were considered. The vertical loading was checked against the design strength of the piles per ft length of road. The bending moment was assumed to be resisted by the inner and outer rows of piles within each cluster with the assumption of no web support. Only the steel was assumed to resist the force; the grout was assumed to only gather and transfer the soil load to the steel.

85. By this process, the required pile density (the number of piles per longitudinal foot of structure) may be computed for the upslope and downslope pile clusters. The resulting pile pattern (for the type 2 structure) is shown in Figure 74.

Instrumented piles and construction

86. Figure 75 shows a plan and profile view of the pile layout with the depth location of the gages indicated. On three of the piles (WES No. 627, 623, and 623) the gages were located at two depths. On the remaining four piles the gages were located at three depth ranges, i.e., between the depths of 5-6, 12-15, and 24-27 ft. On two of the piles (WES 622 and 626) the gages were placed at 90-deg intervals around the pile to monitor four directions: north, east, south, and west. The remaining piles had gages oriented only in the north and south directions, i.e., downslope and upslope, respectively. This plan resulted in 48 data stations, each of which was recorded separately. Table 1 gives a summary of the gage locations.

87. In August 1977 work was begun on the project. The outside lane of the road was removed and excavated 3 ft into the subsurface. A mud mat was poured and served as a working platform for the subsequent pile placement. The entire project required the placement of some 700 cast-in-place piles. The depths of the piles ranged from 40 to 60 ft in depth, the upslope piles extending to the planned depth (for the type 2 section, 40 ft) and the downslope piles extending to the vicinity of rock contact (approximately 60 ft for section 2). It is of interest to note that the presence of the

underlying rock (shale) was not a factor in the design of the repair. After each pile was drilled, the hole was filled with grout and a single No. 10 rebar was inserted into the grout. The project was divided into the X, Y, and Z sections. Following the completion of the pile placement in each section, a reinforced pile cap was poured. In early December 1977, all three sections were completed. Pavement was replaced over the cap beam and the road reopened to traffic.

88. Photographs taken along the road during construction are shown in Figure 76. Both photographs show the open inside lane and the outer lane removed for construction. The construction space was very narrow. Four small track-mounted drill rigs were used to drill the holes. Figure 77 shows the area where the instrumented piles were to be located. The depth to which the outer lane was excavated is indicated in Figure 78.

Strain data

89. Following pile installation, readings were taken and recorded by NYSDOT personnel in the field; the data sheets were sent to WES periodically. At WES the data were transcribed onto magnetic tape for processing and plotting. The basic type of data presentation is a time history plot comparing the output from all the gages along a single pile. These plots are shown in Figures 79-87. They are arranged in numerical order by pile number, starting with pile 622 and ending with pile 628. The first reading of each gage was made on the day of installation. The day that the pile cap was poured is indicated by the large pointer.

90. The output records from the gages were subjectively grouped into two classifications, those gages active for the full period and those for which the gages failed prematurely. Of 43 gages, 24 remained active for the 1-1/2 years they were monitored. During the same period 24 failed; one being damaged during placement. Of the 24 that were active, 21 yielded reasonable data. Three of the gage records were questionable. Of the 24 that had failed, 15 appeared to yield reasonable data or indicated a reasonable trend prior to their failure. Table 2 summarizes the record of gage performance.

91. The strain data were used to calculate the appropriate stress by the elastic relationship, axial stress equals axial strain times Young's modulus.* The stress-time histories plotted from the data are shown in Figures 88-96. The same comments regarding gage performance (Table 2) apply to the stress-time histories.

* Young's modulus of steel used in calculating stress was 30×10^6 psi.

92. One feature which is consistent with all the data is the general relationship of gage output to the placing of the concrete cap. For example, see Figure 82. Gage output steadily increased in either tension or compression following pile installation but before construction of the pile cap. Shortly after the pouring of the cap beam, the gage response diminished and approached a constant level (in most cases).

93. Whether the gage output represents actual pile response to lateral loading during the initial time period prior to placement of the cap beam has not been proved. In some cases regarding strain-gaged reinforcing bars in concrete piles, an initial tensile gage output which diminished with time has been observed in situations where the pile should have been essentially unaffected. For example, such behavior was observed during placement of the instrumented rebars described in Reference 8.

94. Studies⁹ were made to address the phenomenon, without conclusive proof as to the cause. Therefore, a small study¹⁰ was made at WES prior to preparing the reinforcing bars for use in both the California site and this site. The evidence of this study indicated that the curing of the grout did not affect gage output. The piles were surrounded by a stable soil and little gage output was observed. The output that was observed in Reference 8 was therefore attributed to a real output related to the surrounding expansive clays. An undocumented case of similar gage output was also found in which instrumented rebars in concrete surrounded by expansive clays recorded initial tensile stress.

Slide repair performance

95. The NYSDOT monitored the slide repair performance at various locations along the retaining structure. The devices installed included slope indicators and tilt plates. Water level readings were obtained from slope indicator installations. Of particular interest are the data obtained from the slope indicator hole (DHC-5) placed at station 50+00 within the area of the pile cap. The location of this slope indicator is shown in Figure 71. Another slope indicator, DHC-6, was installed approximately 25 ft east of the end of the recently constructed Sta-Wal at station 50+50. The data from these two slope indicators, plotted as depth versus deflection, are shown in Figure 97. The readings shown are from the start of construction to April 1979. (The cut off date of April 1979 was chosen since movements had essentially ceased by that time.) The slope indicator movements plotted as deflection versus time for selected depths and water level elevations versus time are shown in Figure 98.

96. The slope indicator data presented in Figure 97a shows the development of a shear surface between depths of 18 and 24 ft at station 50+00. The maximum deflection recorded was 5.0 in. at the 18-ft depth. The slope indicator at station 50+50 (DHC-6) recorded a maximum deflection of 2.2 in. at the surface as shown in Figure 97b; however, a clearly defined shear surface is not as apparent as at station 50+00.

97. The time histories (Figure 98) show that the movements markedly decreased after the installation of the pile cap. This relative lack of movement is also consistent with the decrease in strain gage output following the placement of the pile cap. The water level recordings, also shown in Figure 98, show an increase in water level during the spring (1979) months. However, this increase in water level did not cause an associated landslide movement.

Discussion

98. The recordings from the instrumentation installed at the California site did not reveal any conclusive evidence related to the possible modes in which the repair reacted to postconstruction loadings and movements. The recorded postconstruction strains in the rebars were simply too small to establish apparent trends. Apparently, the slope (at least in the area of instrumentation) had stabilized prior to construction.

99. The strain gage recordings at the New York site did yield noteworthy information. At the time (1975) this road was being monitored by the NYSDOT survey, the slide movement was reported to be between 0.5 to 0.9 in. per year. The Sta-Wal was constructed as a temporary prevention measure. During the spring of 1977 (when the contract for the pile system was let), heavy rains and snow thaw created a potentially dangerous situation. The slide rate monitored just after installation of the slope indicators (at the start of pile construction) indicated the rate had increased to 3 in. per month. Therefore, this particular project presented an excellent opportunity to make measurements in a kinetic environment.

100. In retrospect it is unfortunate that more strain gage instrumentation was not installed at this site and that a more thorough collection of subsurface data were not obtained. Also, in light of the ICOS design concepts, additional consideration should be given to instrumentation to monitor bending and loads in the pile cap.

101. The postconstruction loads acting at the strain gage locations are schematically shown in Figure 99. For clarity, piles 623 and 627 are shown beside piles 626 and 622; however, in profile they are coincident. The axial load in each rebar at the gage station is (theoretically) the average of the individual readings at the gage station. The average axial load at each gage station (postconstruction) is shown in Figure 100. Ideally, the figure represents the loads within the "composite beams" formed by the upslope and downslope pile clusters. The design shear plane is also shown in Figure 100. The slope indicator data given in Figure 97b verify that the location of the design slide plane was essentially correct at least as to the depth of the maximum deflection. The slope indicator data do not indicate a single abrupt shear plane, but a rather wide shear zone extending from depths of 18-28 ft.

102. The loads within the composite beam due to pure bending may be calculated by deducting the average axial load from the outer fiber (outer rebar) loads. Figure 101 shows the interpreted bending loads and axial loads acting on the upslope and downslope "composite beams." There are an insufficient number of data points to accurately define the "beam" behavior; however, the load interpretation represented in Figure 101 is entirely reasonable when compared to the slope indicator data. Qualitatively, the interpreted bending loads would yield a pair of beams bent in the shape shown in Figure 102. This schematic interpretation is consistent with design considerations except for a reversal of the expected bending in the upslope cluster. Apparently, the forces in the downslope cluster (which includes tensile axial loads in the upper portions) are transferred through the pile cap to the upslope cluster causing the upslope "beam" to be pulled into the material between the pile clusters. The upslope "beam" also does not experience appreciable axial loading except for a tensile load in the vicinity of the design shear plane.

103. The axial "beam" loads in the downslope cluster indicate tension within the upper 10-12 ft of the structure and compressive axial loads elsewhere. The design axial load on the downslope cluster (see Figure 73b) of 2020 lb per ft of the structure should produce an axial loading in the "beam" of 10,250 lb (i.e., each 5 ft of structure is supported by one downslope cluster). The interpreted axial load of 21,000 lb is not considered to be inconsistent in light of the absence of axial loading on the upslope cluster and the lack of definition of the shear plane.

PART IV: CONCLUSIONS

104. The installation of strain-gaged rebars within the cast-in-place piles of both the California and New York sites proved to be a useful method for monitoring the loads within the two repair structures. The instruments responded to foundation movements and loadings satisfactorily. At the California site (the major site) the instruments indicated that the resultant foundation loadings were quite small; i.e., the loadings were on the order of loadings caused by construction operations. There was no indication that landslide movements occurred after installation of the repair. Whether landslide activity at this site will reoccur in the future is a matter of conjecture.

105. The strain gage instrumentation at the New York site did indicate loadings due to landslide activity. The instrumentation for this site was not nearly as extensive as at the California site; however, the few gage stations that were installed did provide meaningful measurements.

106. The design approach taken by the designers at the two sites differed. For the California site, the repair was visualized as a buried retaining wall; the concept being to provide additional shear capacity (by the casting of piles) so as to increase the factor of safety to a stable value. The designers of the New York repair work approached the design from a structural viewpoint and analyzed a space frame structure acted upon by active earth forces.

107. The relevance of the design to the subsequent response of the repair structure at the California site cannot be ascertained due to the lack of instrumentation responses. The abbreviated instrumentation program at the New York site did yield information that is consistent with the design approach for that repair. Pertinent facts associated with the New York site are:

- a. The repair was installed within an area in which landslide movements were currently active.
 - b. Movements continued at the site during the casting of the piles but before the placement of the pile cap.
 - c. Movement essentially ceased following the placement of the pile cap.
 - d. The various instrumentation (strain gages and slope indicators) gave responses consistent with the designers concepts.
- Results of the stone column tests (see item d under "Purpose and Scope") are unrelated to the main purpose of this investigation and are reported separately.¹¹

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5. Engineering News Record. 1976 (11 March), "Small Diameter Pile System Retains Sliding Embankment on Road Bridge," p 14.
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7. Daily report, Soil Mechanics Bureau, New York State Department of Transportation, dated 21 June 1976.
8. Ehr Gott, J. Q. 1977. "Summary of Installation of Test Pile on I-55, Jackson, Mississippi, In-House Progress Report 1," U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.
9. Alexander, A. M., and Ainsworth, D. L. 1971. "Strains Developed in Concrete During and Subsequent to Hardening," Miscellaneous Paper C-71-6, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.
10. Ehr Gott, J. Q. 1977. "Evaluation of Earth Strain Gage Response When Embedded in a Grout Pile, In-House Progress Report 2," U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.
11. Ehr Gott, J. Q. 1977 (29 April). "Field Shear Test on Stone Columns," Memorandum for Record, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

Table 1
List of Piles and Gage Locations

WES Pile No.	No. of Gage Stations with Depth	Depths of Gages ft			No. of Gage Bridges Around Pile at each Depth Station	Direction of Gages Around Pile
622	3	6	13.5	27	4	Downslope and 3 others at 90, 180, and 270 deg*
627	2	6	13.5	--	2	Downslope and upslope only
625	3	5	12.5	25	2	Downslope and upslope only
628	2	--	15.5	24	2	Downslope and upslope only
624	3	6.5	15	24	2	Downslope and upslope only
623	2	--	15.5	26	2	Downslope and upslope only
626	3	6.5	15	26	4	Downslope and 3 others at 90, 180, and 270 deg*

* Measured from location of downslope gage.

Table 2
Summary of Gage Performance

<u>Active for Full 1-1/2 Year Period</u>		<u>Failure during Monitoring Period</u>	
<u>Reasonable Data</u>	<u>Questionable Data</u>	<u>Reasonable or Slight Trend in Data</u>	<u>Questionable Data</u>
622 CN	625 AS	625 AN	622 AN
625 BS	625 BN	624 AS	622 AE
625 CS	624 CS	624 BN	622 AW
625 CN		623 BN	622 CE
624 AN		622 AS	622 CW
624 BS		622 BN	626 CW
624 BN		622 BS	627 AN
626 AN		622 BE	627 BN
626 AS		622 BW	623 CN
626 BN		622 CS	
626 BS		626 CN	
626 AE		626 CS	
626 AW		626 CE	
626 BE		627 AS	
626 BW		627 BS	
628 BN			
628 BS			
628 CN			
628 CS			
623 CS			
623 BS			

FONDEDILE "RETICULATED ROOT PILES RETAINING
STRUCTURES" TO STOP A LANDSLIDE THREATENING
A BUILT UP AREA.

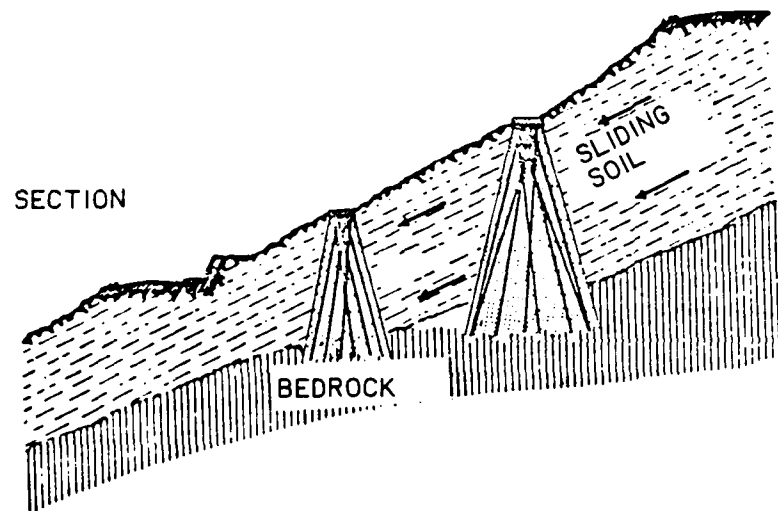
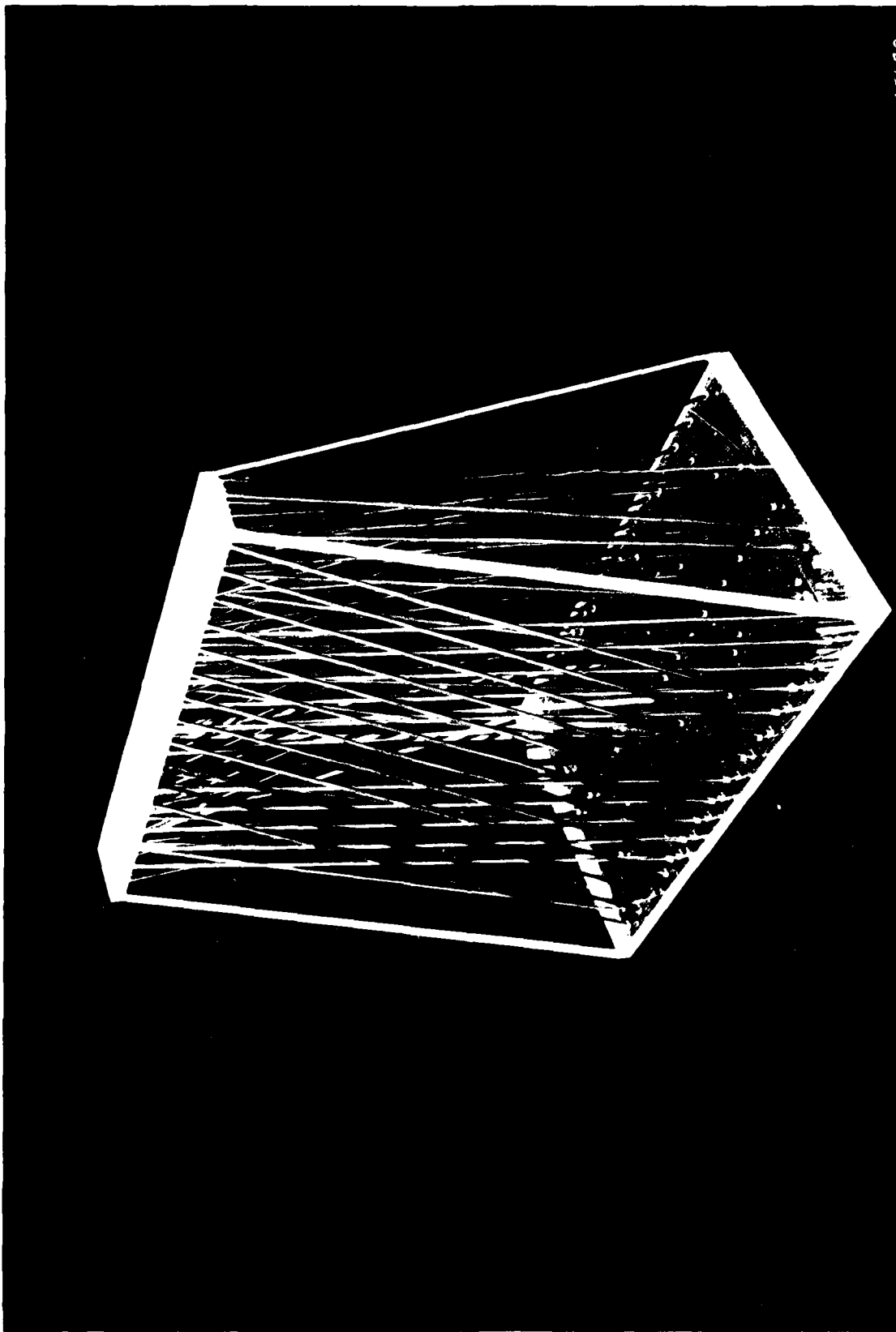


Figure 1. A hypothetical example of the use
of a reticulated pile system for landslide
control (reference 2)



45470

Figure 2. Model of pile systems, California site

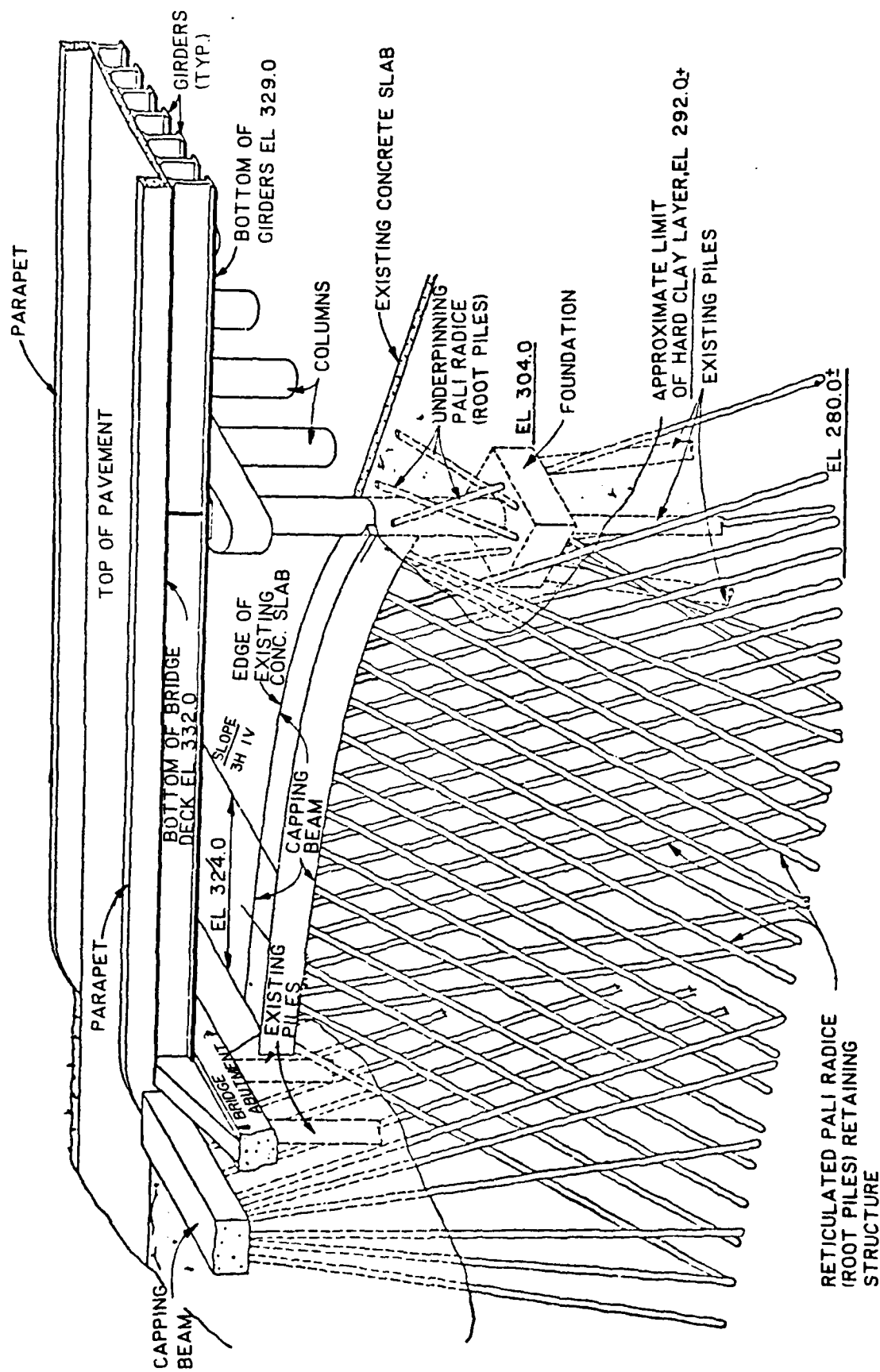


Figure 3. Installation of 8-in.-diam root piles at the Gallatin Street Interchange, Interstate Route 55, Jackson, Miss.

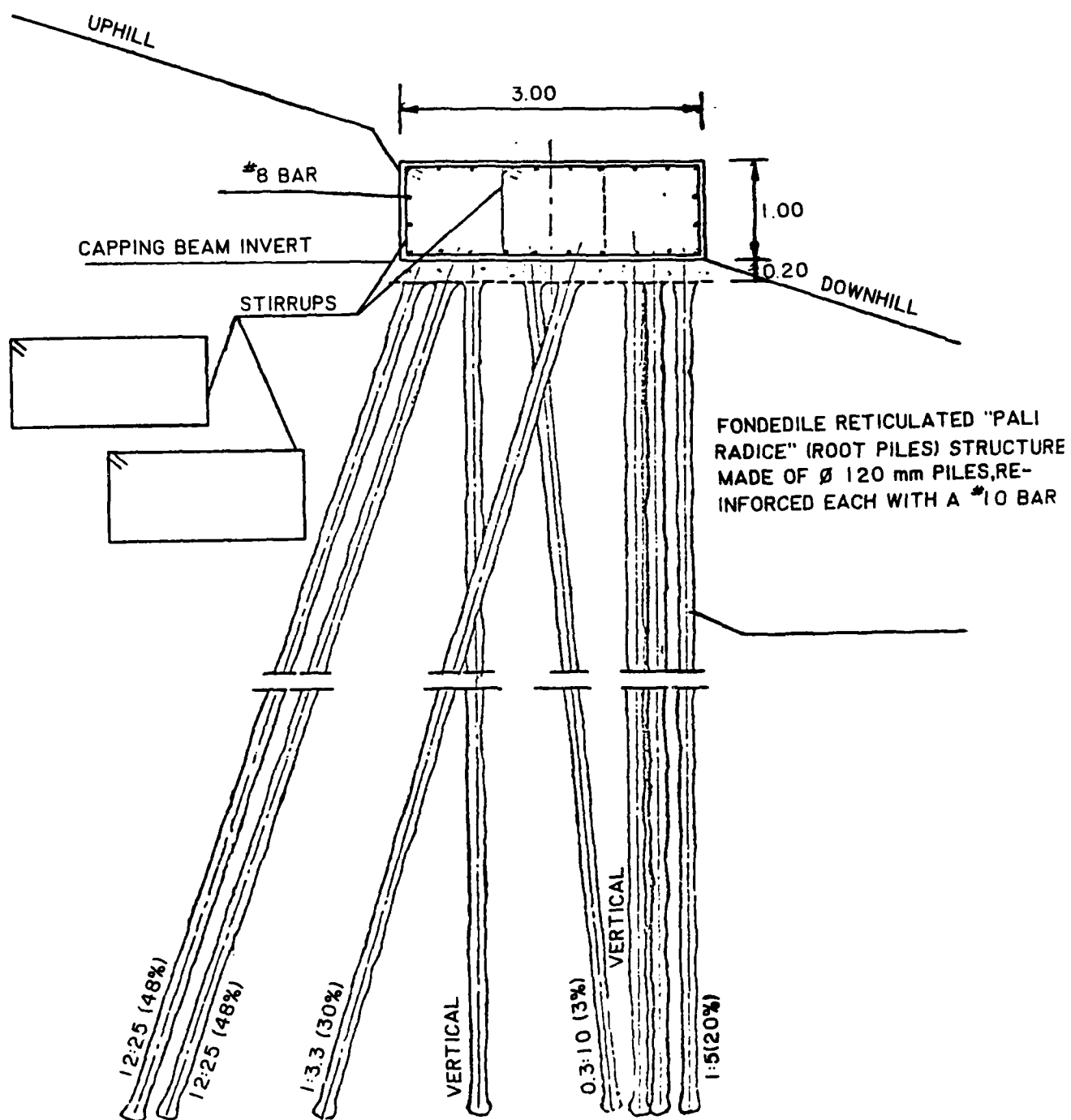


Figure 4. Slope stabilization to protect the foundation of the Savio viaduct over the Tevere River, Rome, Italy (reference 4)

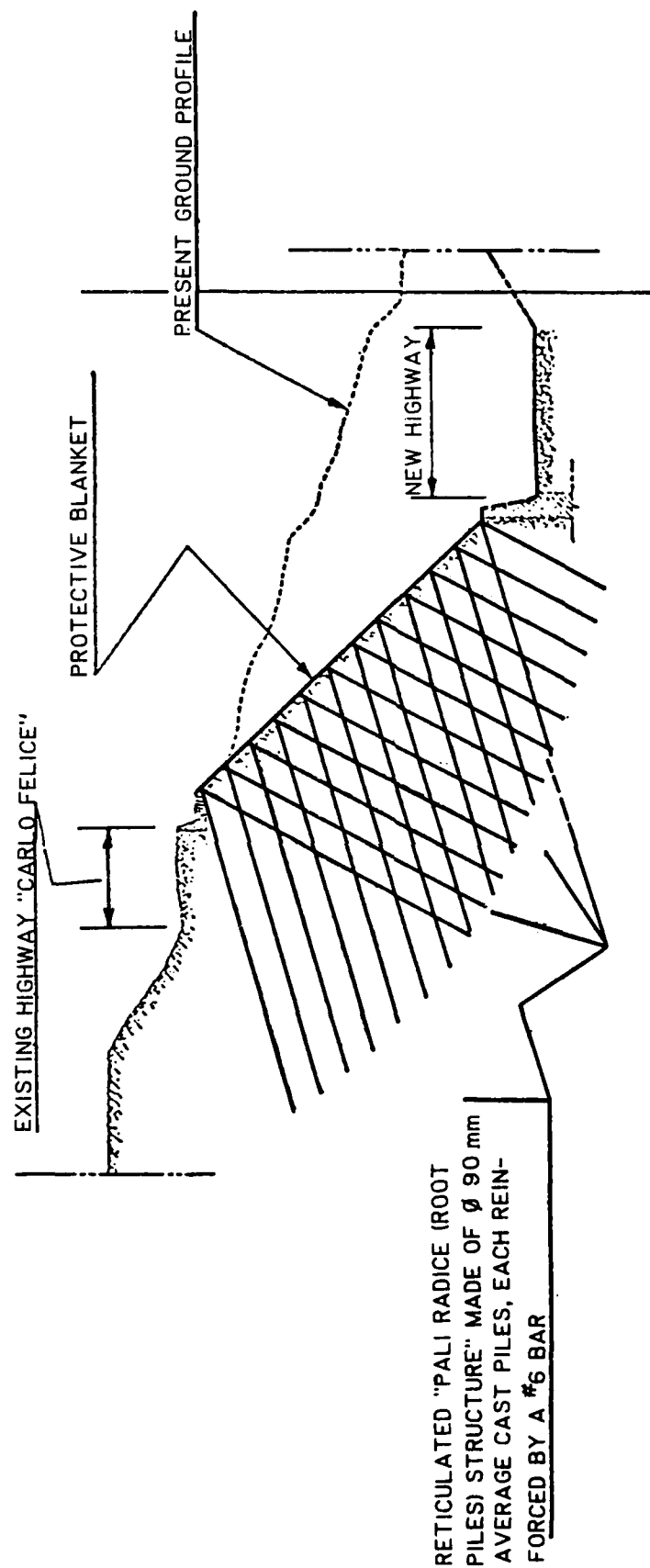


Figure 5. Reticulated pile installation to control a slope whose movement endangered the Carlo Felice Highway in Sardinia in 1965 (reference 4)

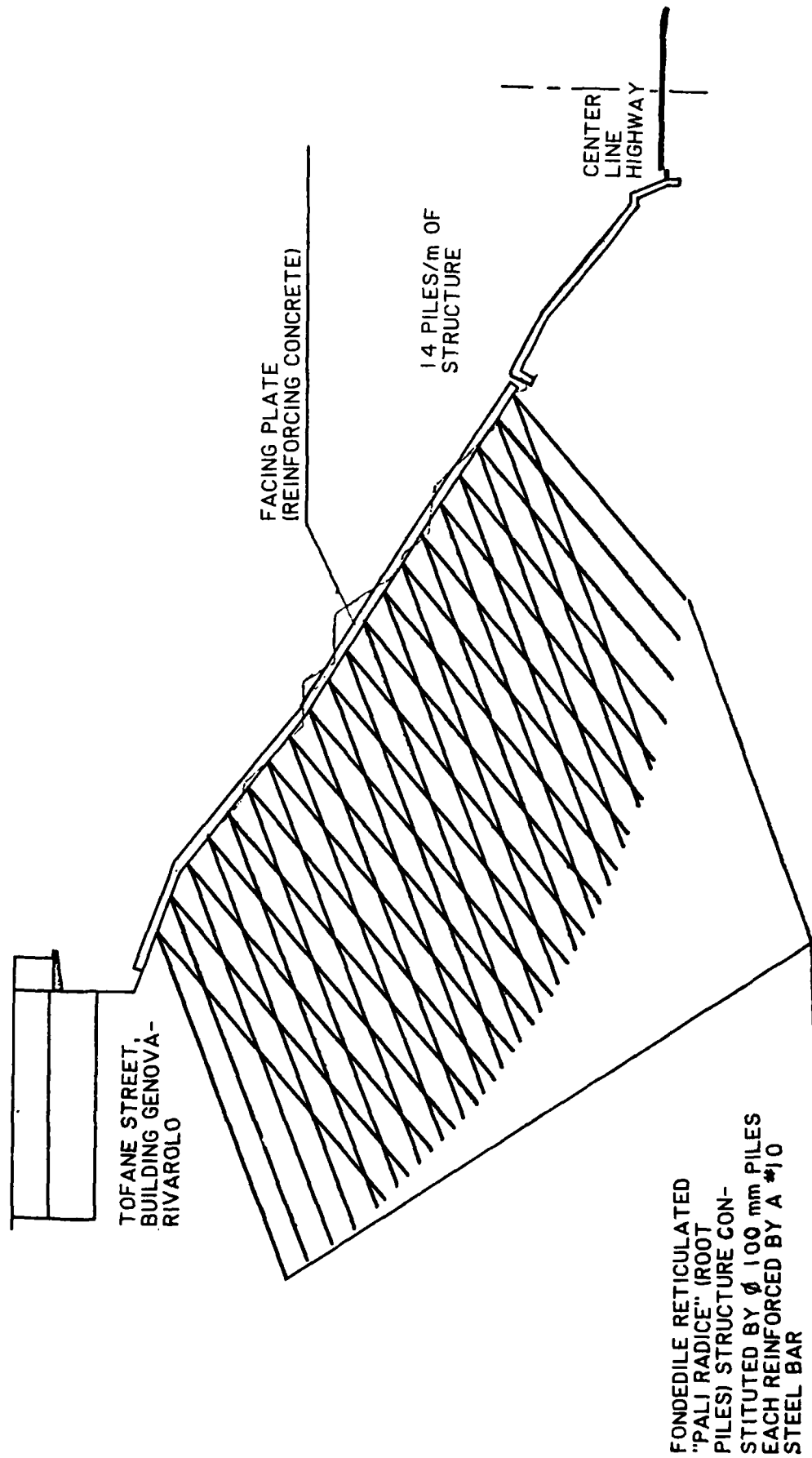
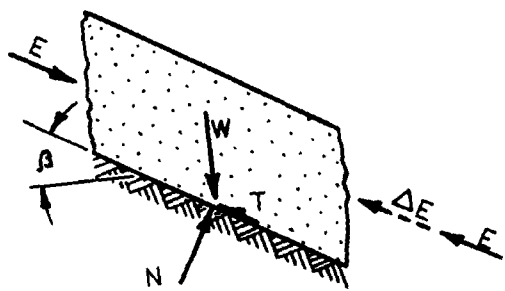
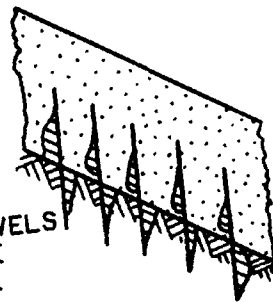


Figure 6. Reticulated piles used to control the movement of a slope in Genoa, Italy, in 1972 (reference 4)

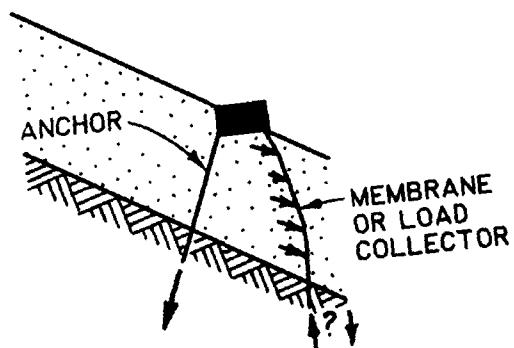


(a) SIMPLE LANDSLIDE MODEL OF A SEGMENT OF AN INFINITE SLOPE

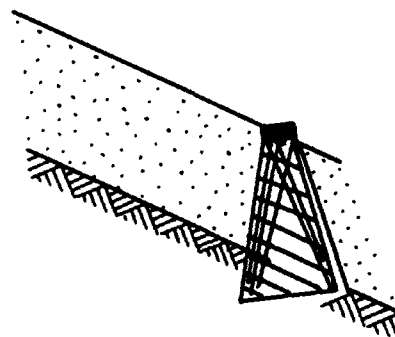
$\Delta T = \text{NO. OF DOWELS} \times \text{ALLOWABLE SHEAR/DOWEL}$



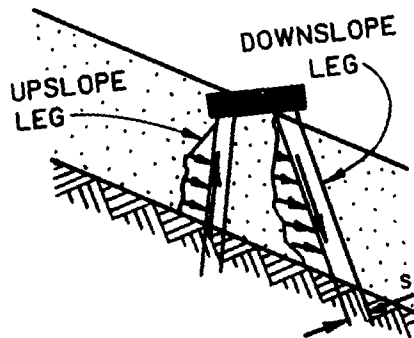
(b) PILES AS DOWELS ACT TO INCREASE THE FORCE T



(c) RETICULATED ROOT PILE SYSTEM ADDING FORCE ΔE VIA MEMBRANE AND ANCHOR ACTION

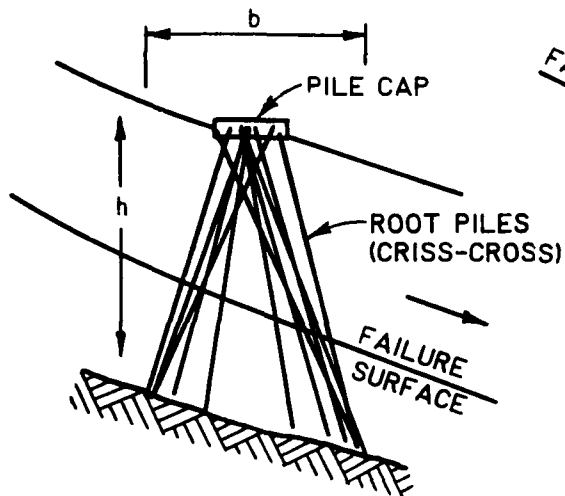


(d) RETICULATED ROOT PILE SYSTEM "...STITCHING TOGETHER THE DIFFERENT ROCK LAYERS, TRANSFORMING THE ENTIRE MASS INTO A KIND OF GRAVITY RETAINING STRUCTURE..."

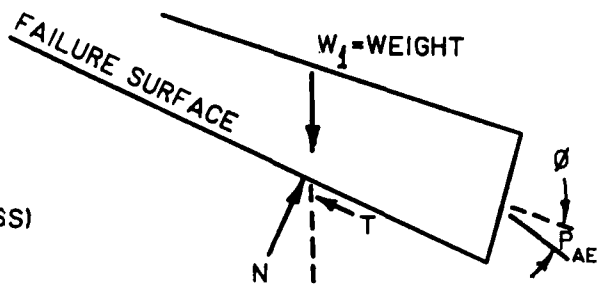


(e) A NONRETICULATED ROOT PILE SYSTEM WHICH FUNCTIONS AS A STRUCTURAL BENT WHOSE "LEGS" ARE RETAINING WALLS

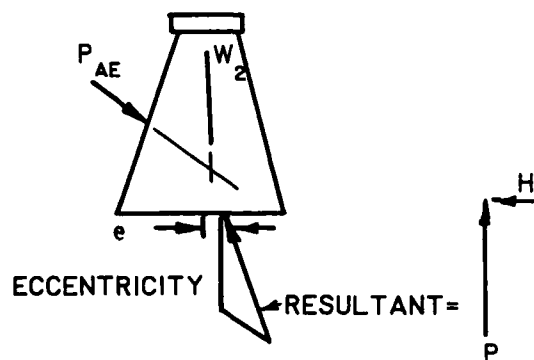
Figure 7. Use of pile structures to control landslides



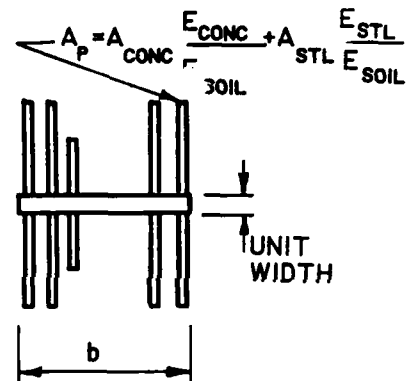
(a) FONDEDILE RETICULATED PILE STRUCTURE



(b) CALCULATION OF ACTIVE EARTH PRESSURE



(c) STABILITY OF "GRAVITY" WALL AGAINST OVERTURNING AND SLIDING



(d) TRANSFORMED SECTION

STEP:

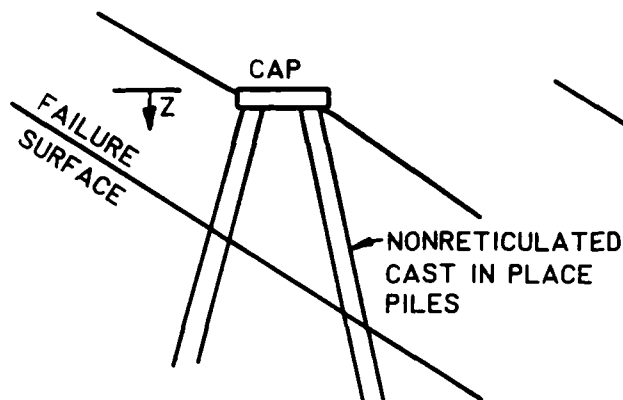
$$\textcircled{1} \sigma = \frac{P}{A_{\text{TRANS}}} \pm \frac{P_e}{I_{\text{TRANS}}} (b/2), \sigma \geq 0 \text{ (ie COMPRESSIVE)}$$

$$\textcircled{2} H < (\text{SHEAR STR. OF SOIL} \times b) + \text{SHEAR CAPACITY OF PILES}$$

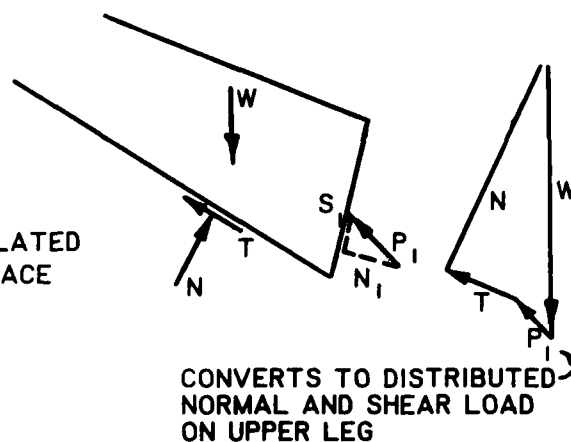
$$\textcircled{3} b/6 \times \sigma_{\text{MAX}} A_p < \text{PILE CAPACITY}$$

(e) DESIGN CRITERIA.

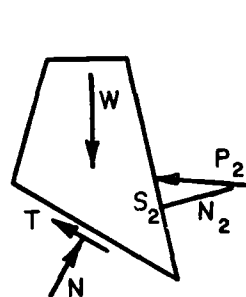
Figure 8. Principles of design from Fondedile literature



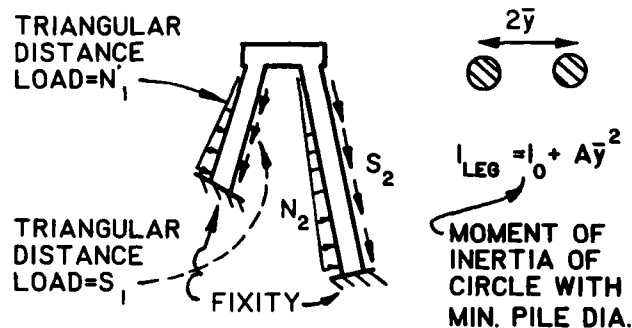
(a) ICOS PILE STRUCTURE



(b) ACTIVE EARTH PRESSURE ON UPPER "WALL"



(c) ACTIVE EARTH PRESSURE ON LOWER "WALL"



(d) STRUCTURAL LOADS AND ASSUMPTIONS

STEP

- (1) Solve for shear (V) and moment (M) using Hardy-Cross Method or Structural Analysis Computer Program
- (2) Compute axial loads in piles = $\frac{\text{Cap wt}}{2} + \int_0^z s dz + M(z) \cdot y$. Note
all bending is assumed to be resisted by the pile axial loads in a manner similar to that in which a truss carries moment.
- (3) Axial load < working strength x area of steel only
- (4) Axial load < pile capacity in compression or tension
- (5) Shear V < working strength x area of steel only

(e) SIZING THE STEEL BAR AND CHECKING ADEQUACY OF THE STRUCTURES

Figure 9. ICOS pile structure design

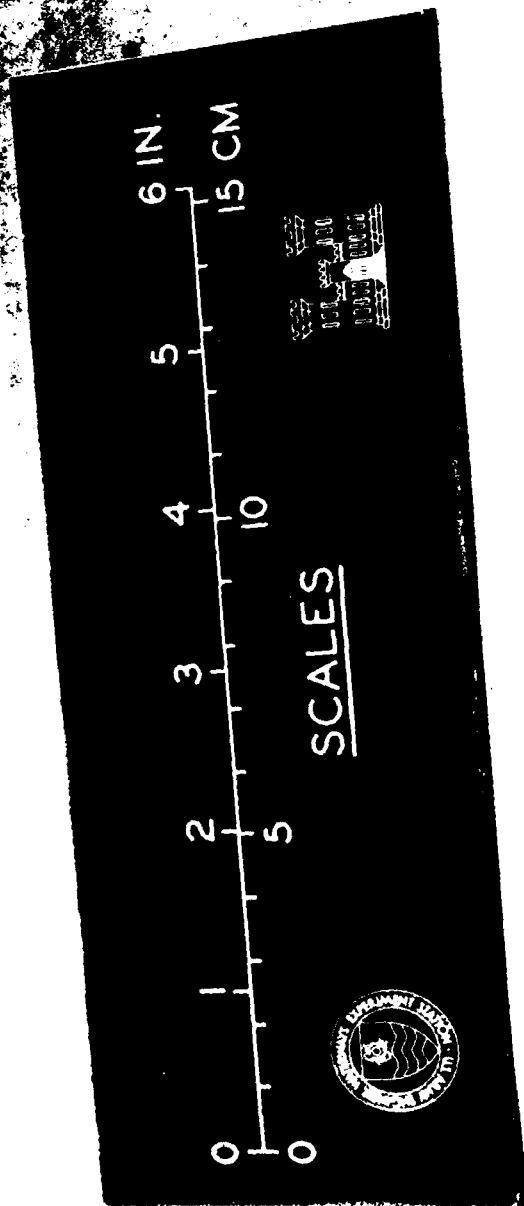
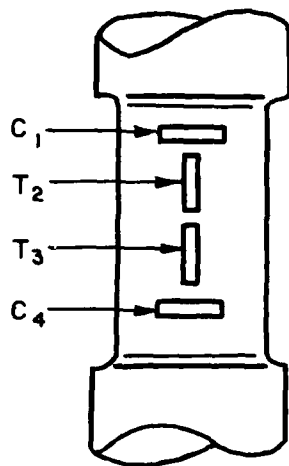
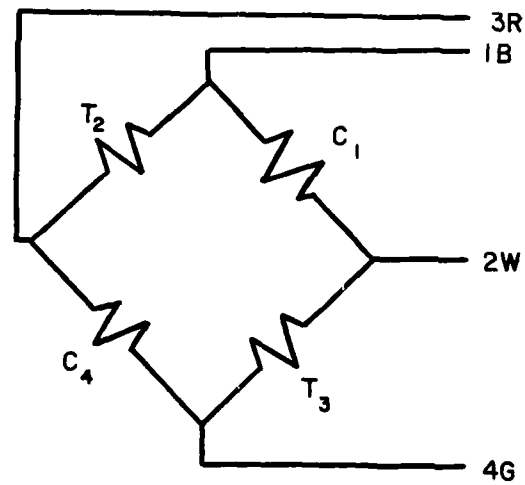


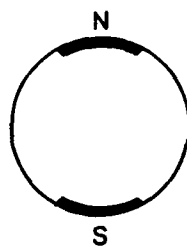
Figure 10. Rebar after lathing, ready for bonding of strain gages (Note punch marks which were placed to insure proper gage orientation)



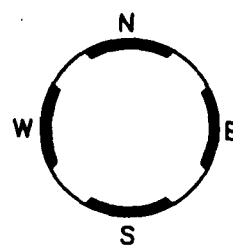
(a) FULL-BRIDGE STRAIN GAGE LOCATIONS



(b) BRIDGE CIRCUIT



(c) TWO-GAGE CONFIGURATION



(d) FOUR-GAGE CONFIGURATION

Figure 11. Details of strain-gaged rebars

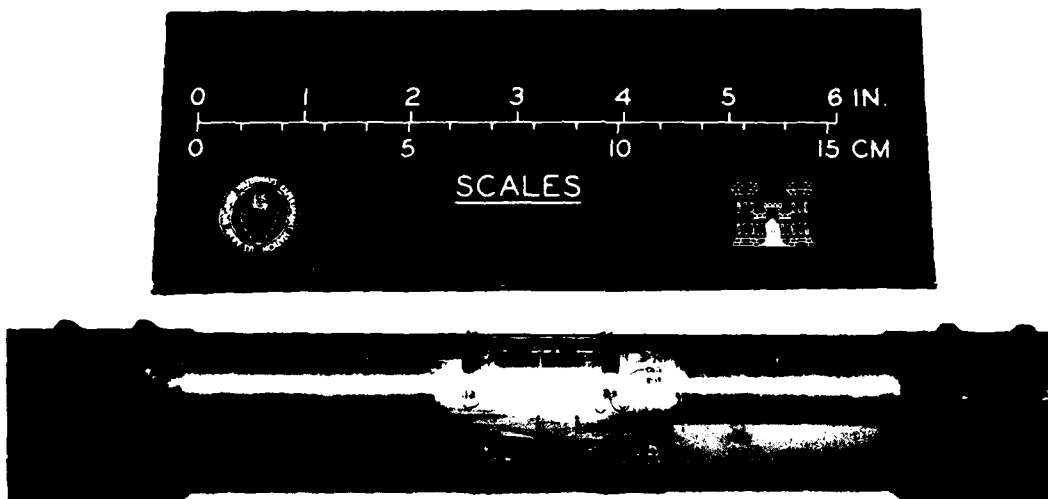


Figure 12. Rebar with a completed strain gage bridge attached

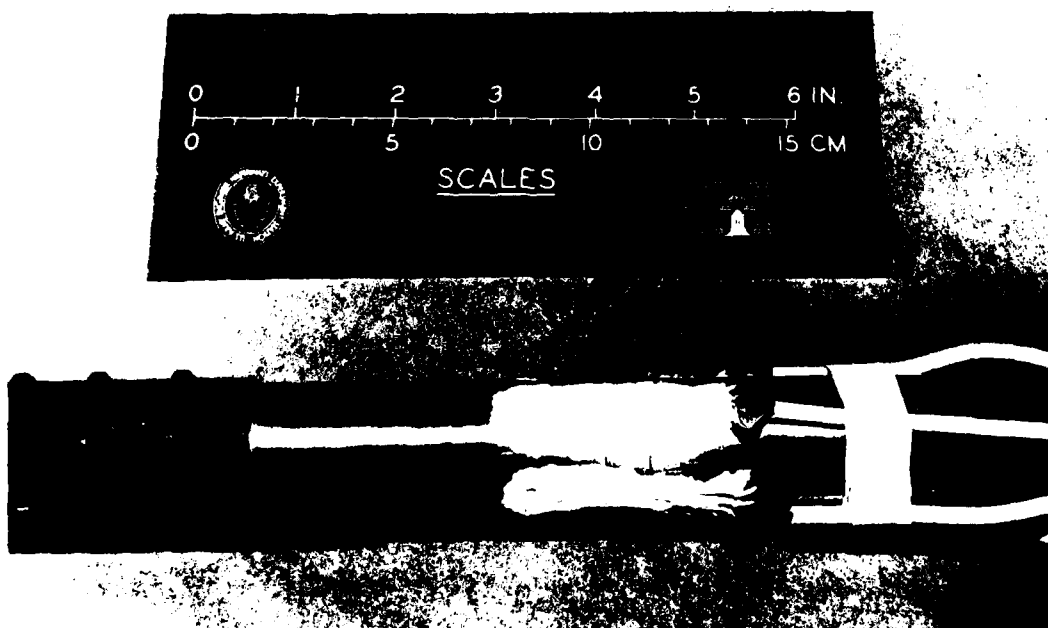


Figure 13. Rebar with first coat of waterproofing applied

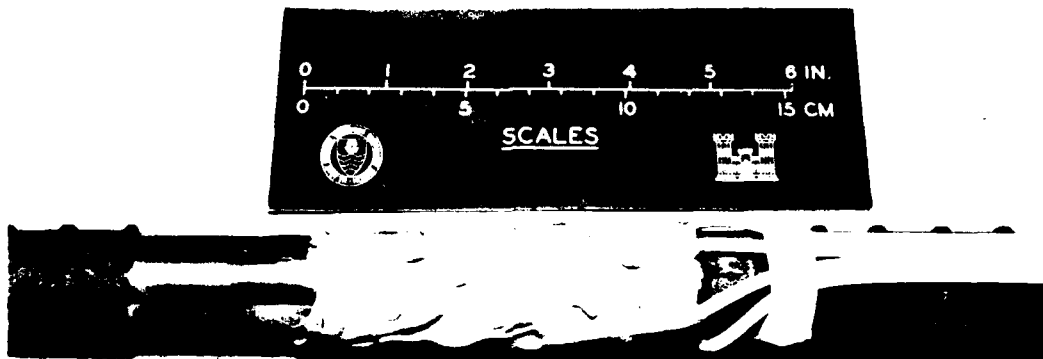


Figure 14. Rebar with final coat of waterproofing applied

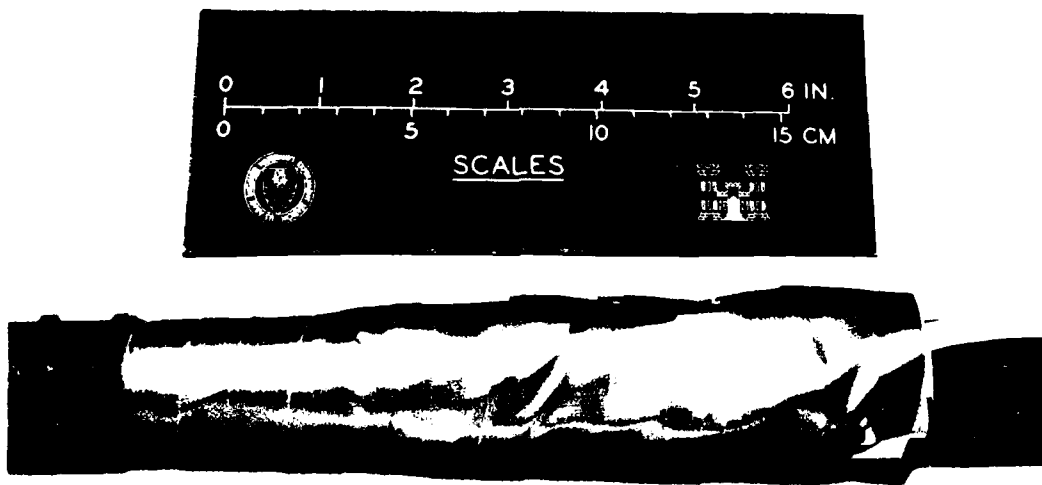


Figure 15. Rebar instrumented and ready for field placement

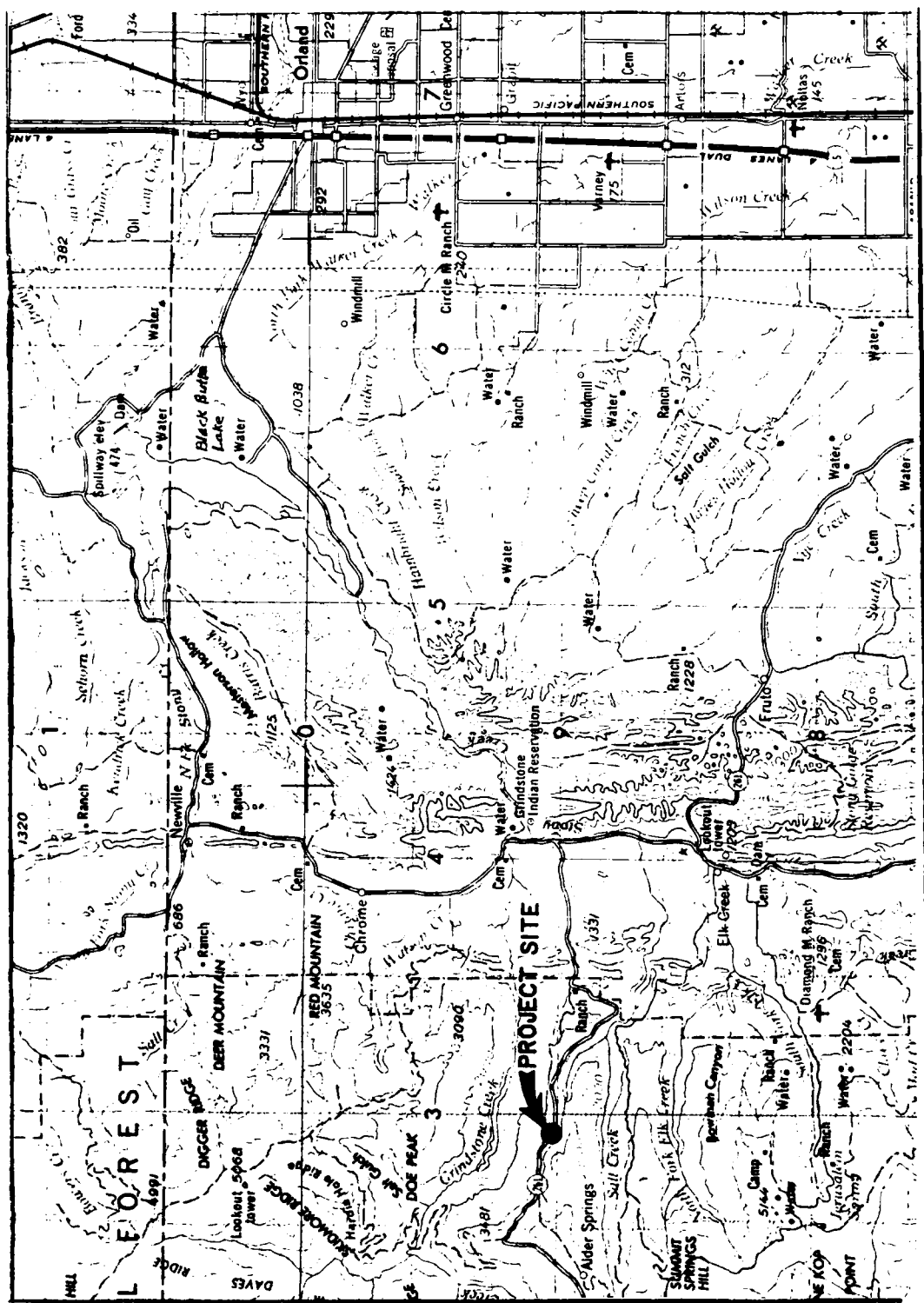


Figure 17. Location map, California site

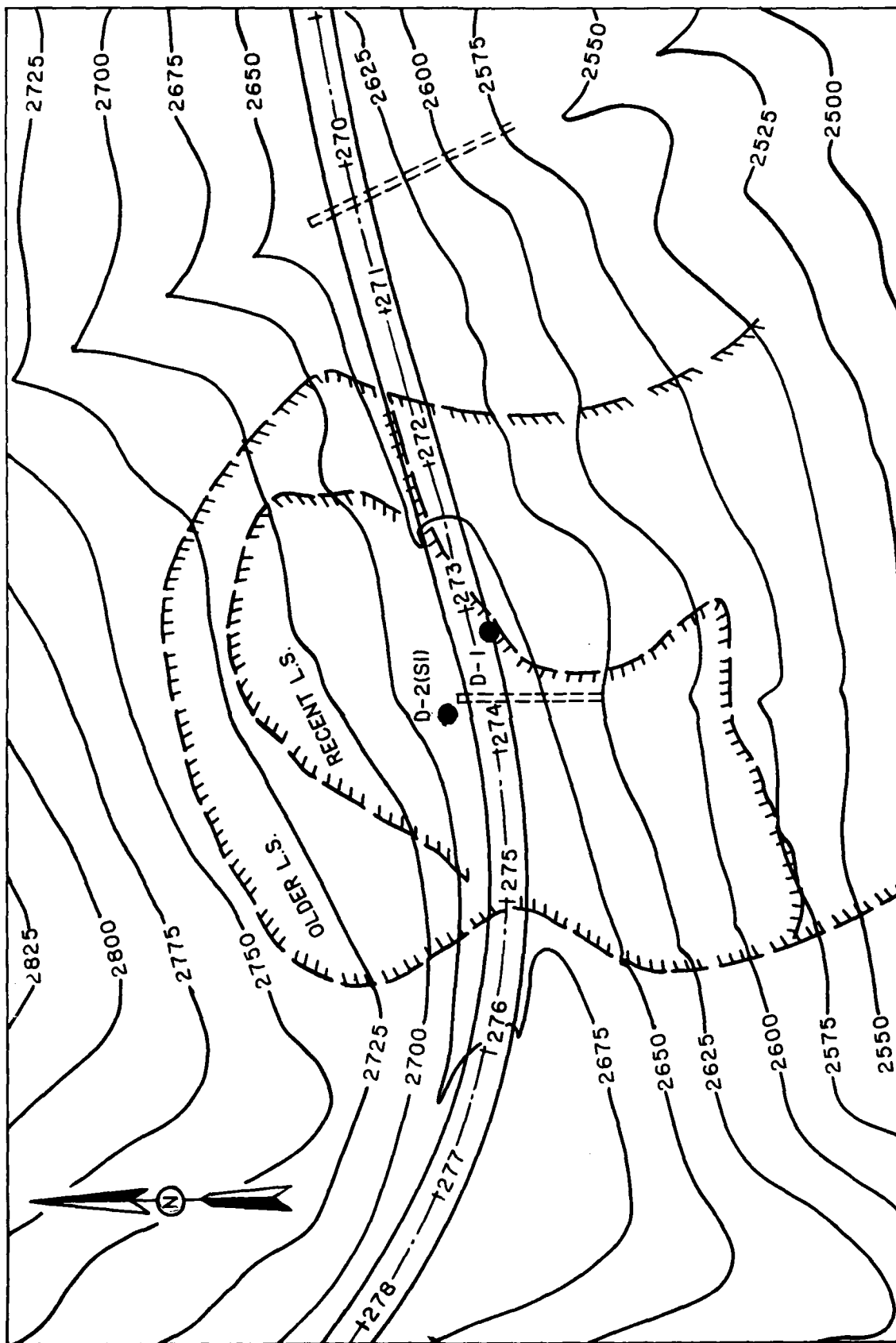


Figure 18. Topography of project site

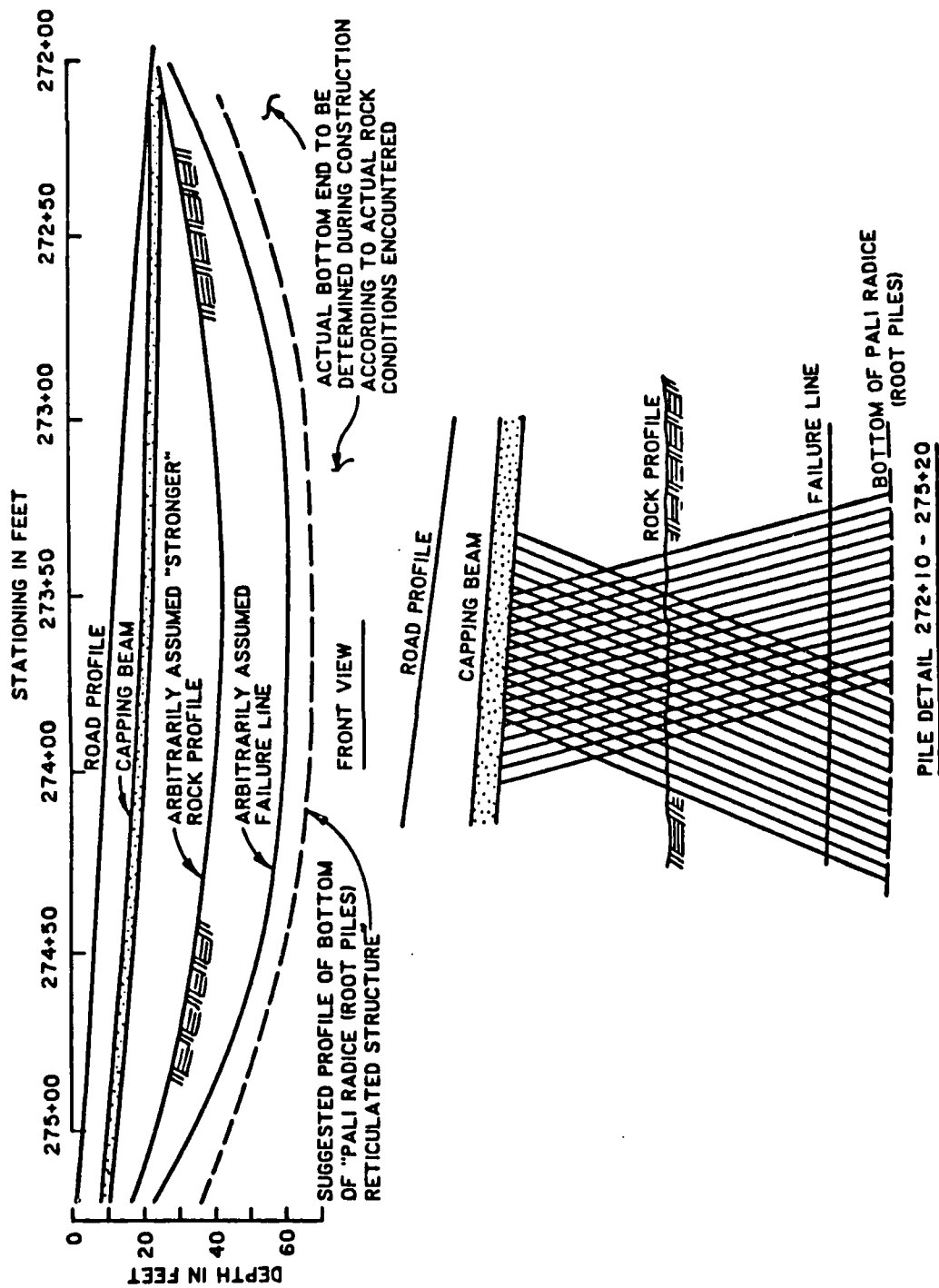


Figure 19. Front view of pile network

LEGEND

4--PILE SEQUENCE NO.

O_D--PILE TYPE

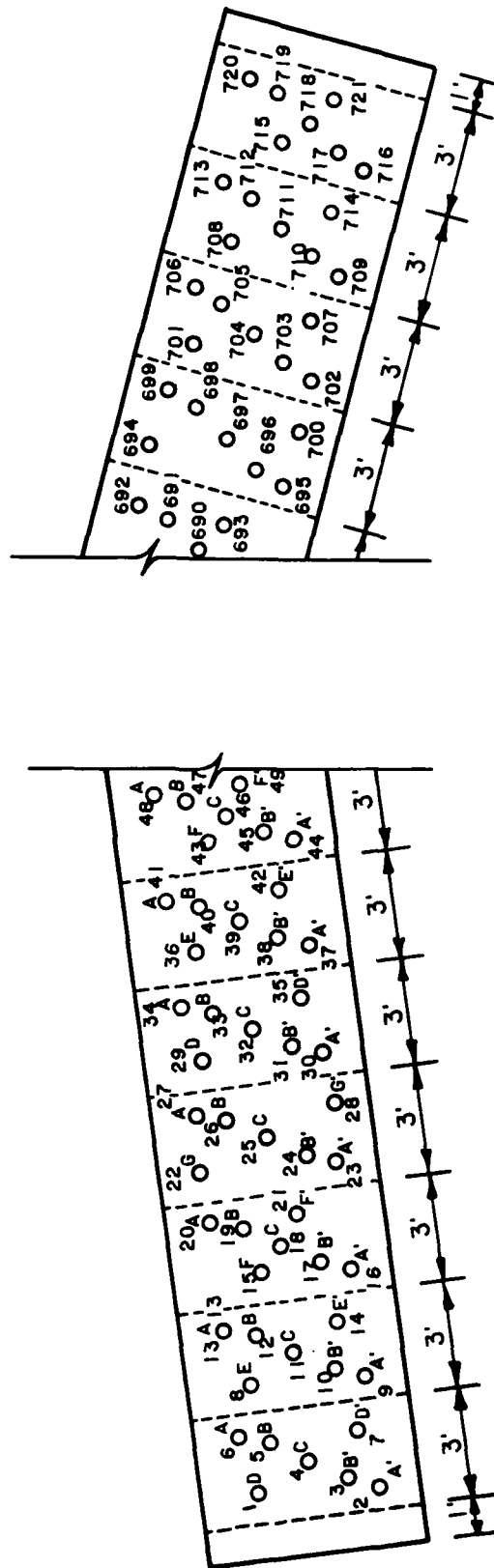


Figure 20. Plan view of pile network

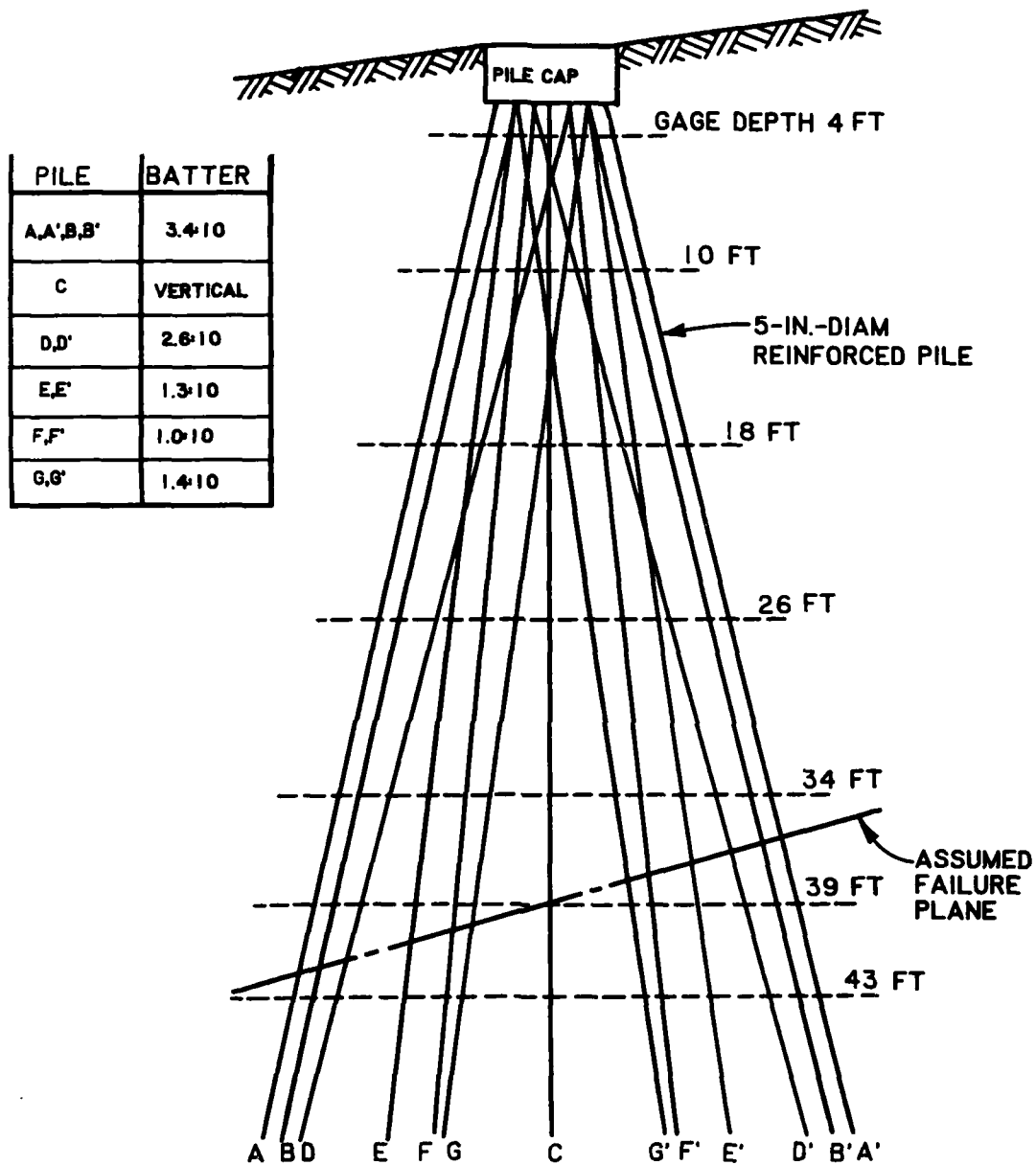


Figure 21. Cross-sectional view of pile network



Figure 22. Pile cap and previously placed plastic sleeves

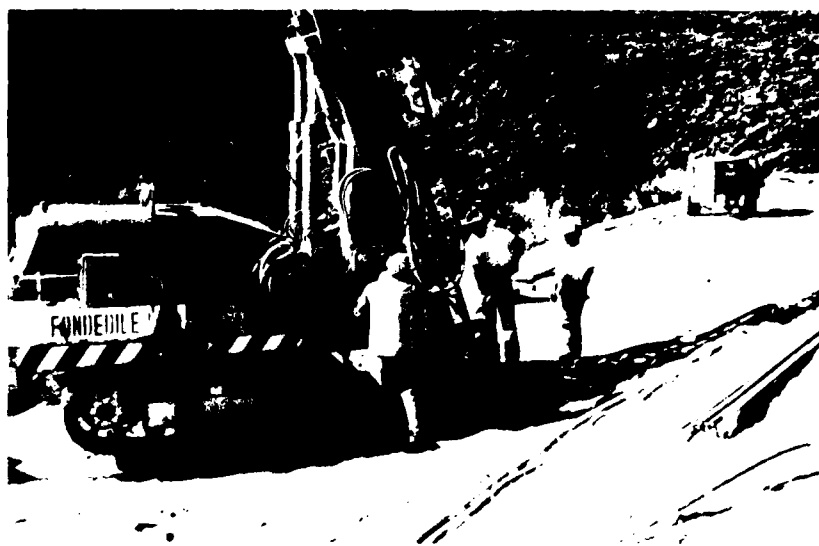


Figure 23. Drilling rig at construction site

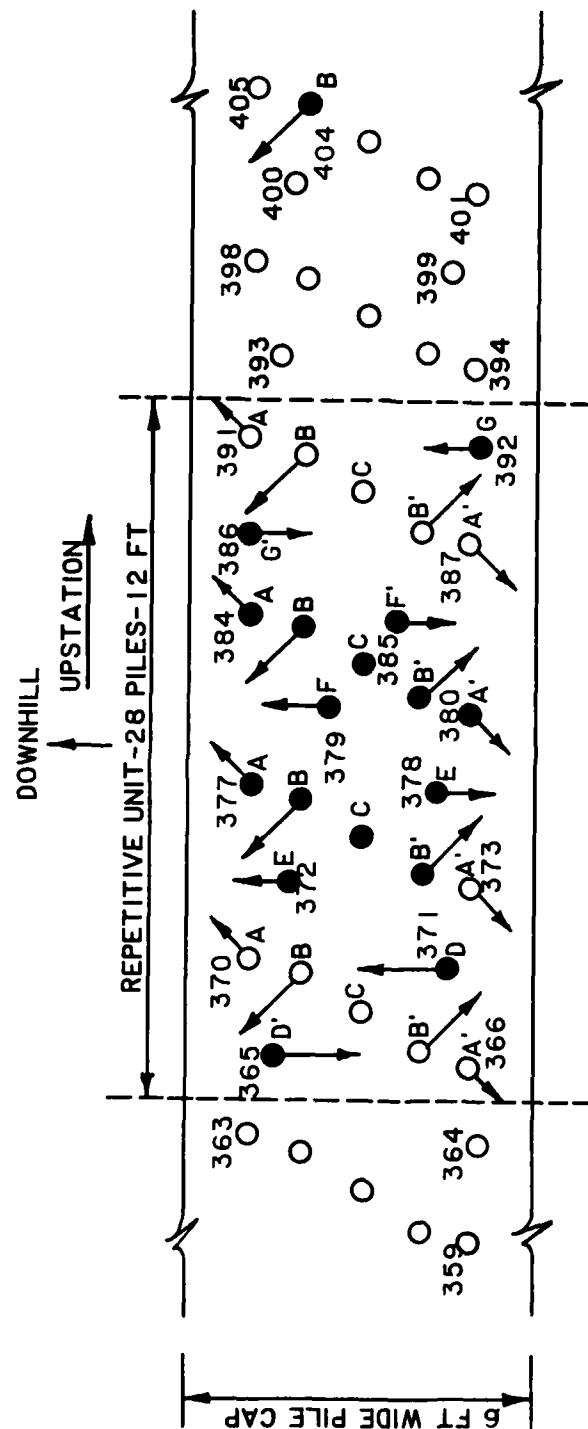


Figure 24. Plan view of instrumented section



Figure 25. Centering devices used on instrumented piles



Figure 26. Channels for placement of electrical cables



Figure 27. Barrels employed for collecting cable ends



Figure 28. Completed gage station (pile cap covered by about 5 ft of fill) is located below the shoulder outside of guard rail

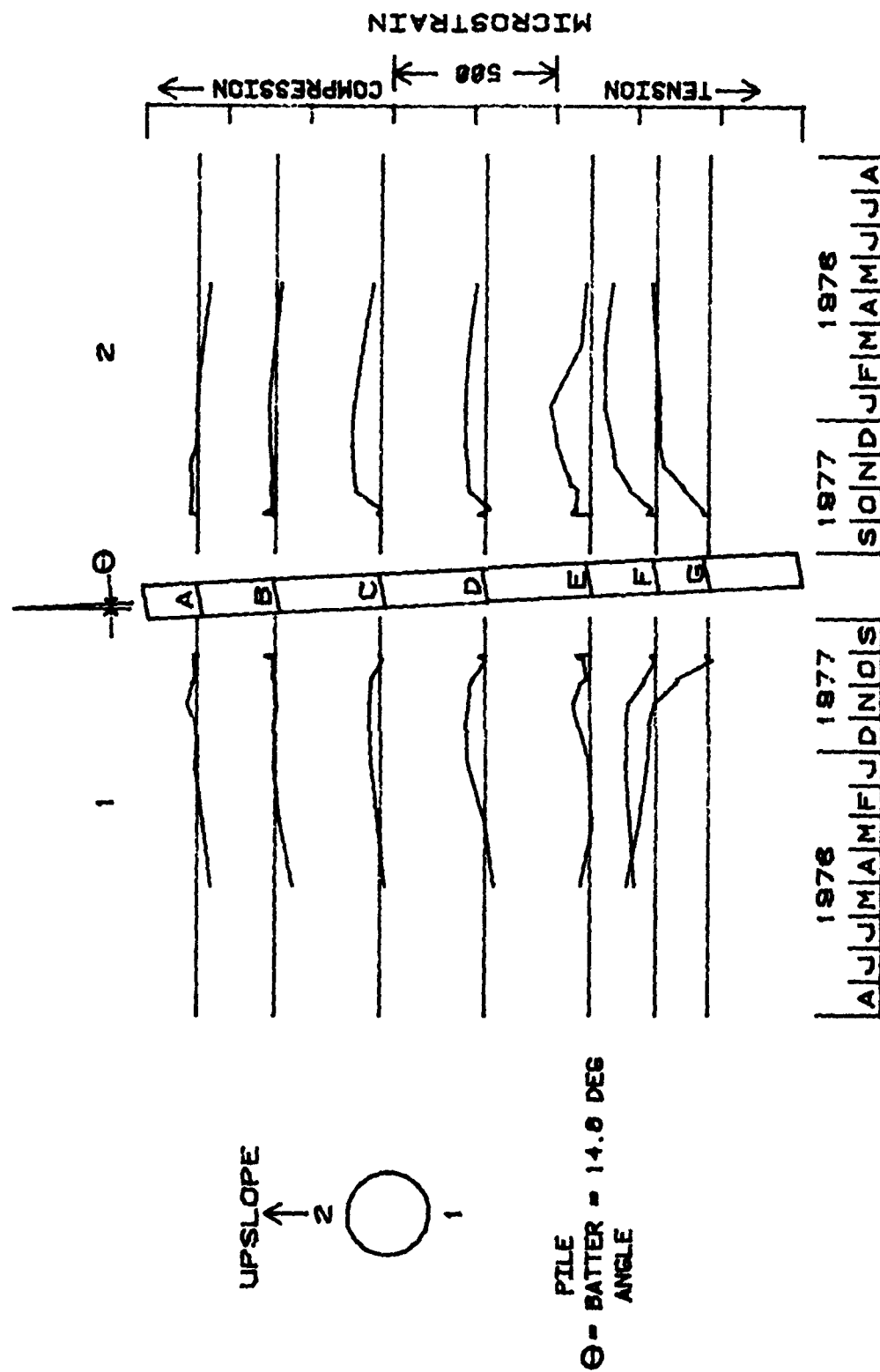


Figure 29. Strain-time histories along the length of pile 365, upslope and downslope

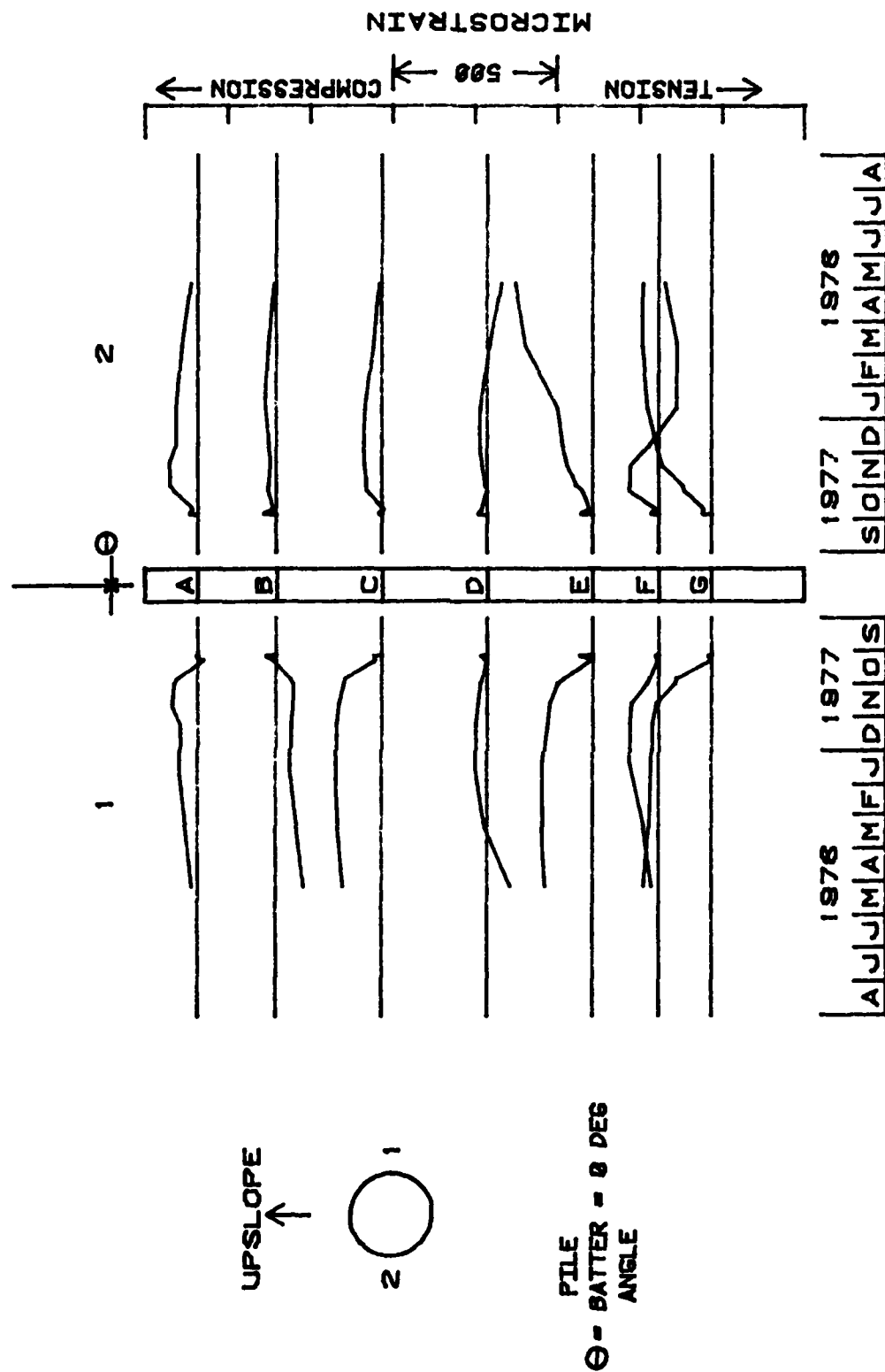
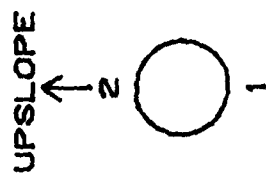


Figure 30. Strain-time histories along the length of pile 365, upstation and downstation



PILE
 Θ - BATTER = 7.4 DEG
 ANGLE

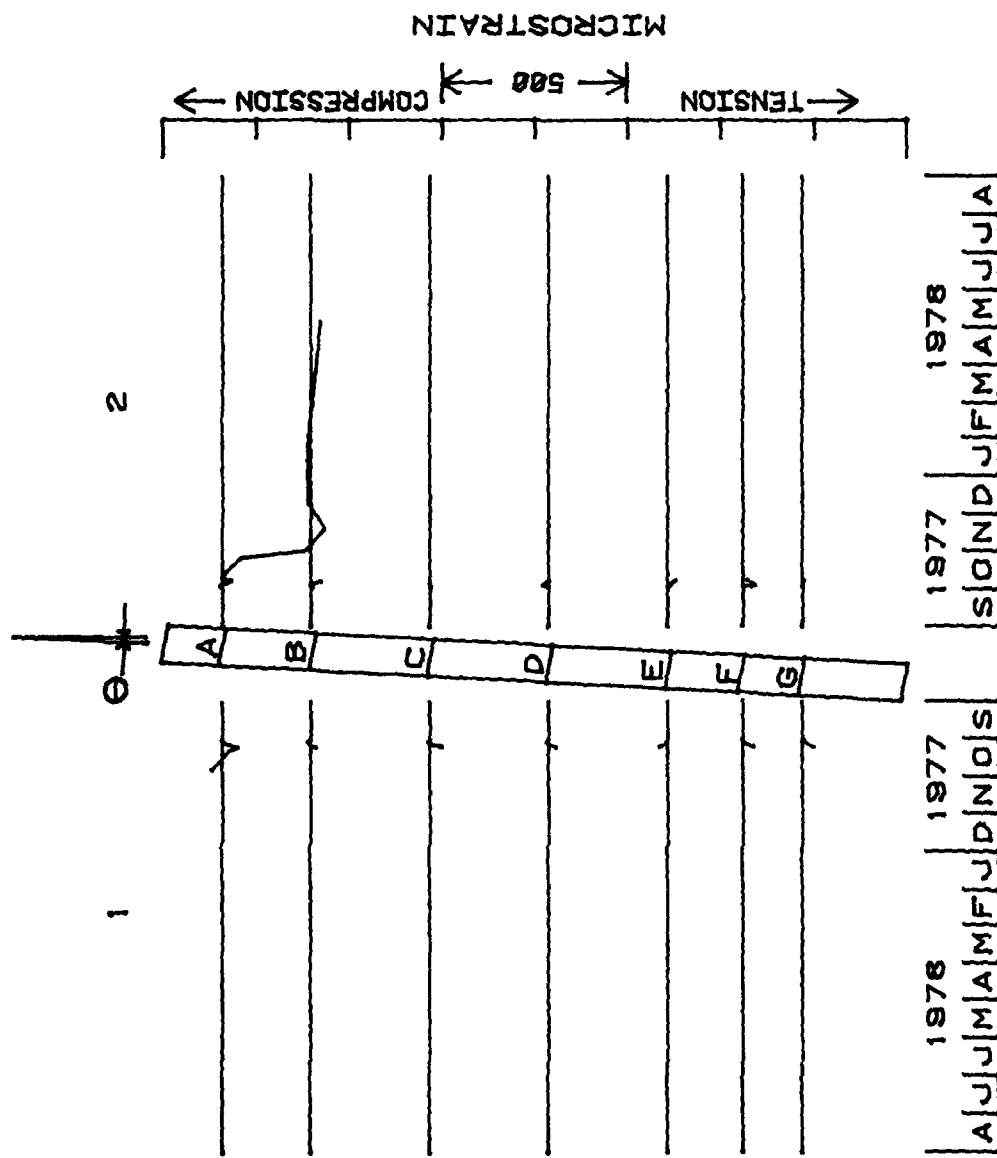


Figure 33. Strain-time histories along the length of pile 372, upslope and downslope

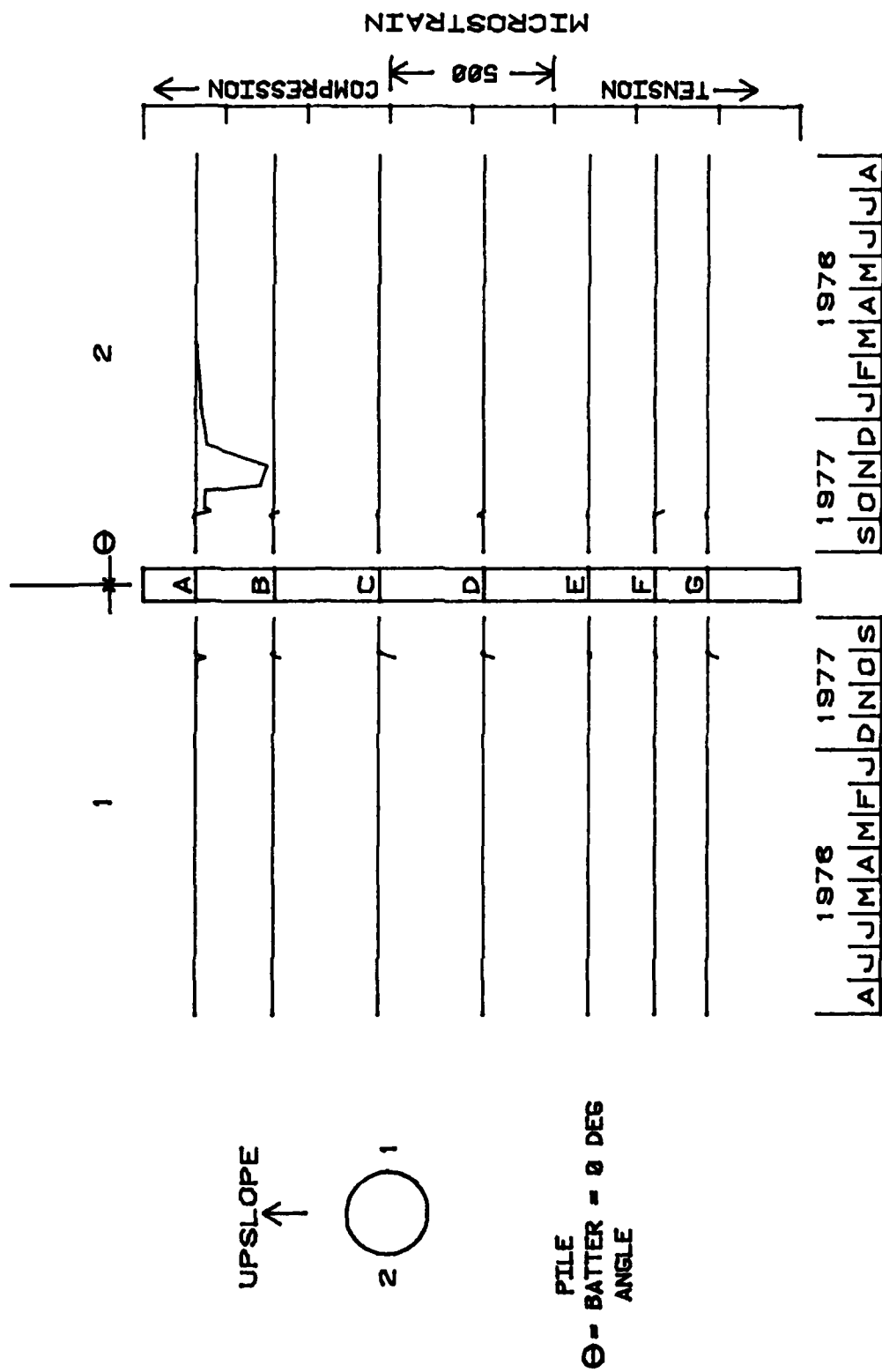


Figure 34. Strain-time histories along the length of pile 372, upstation and downstation

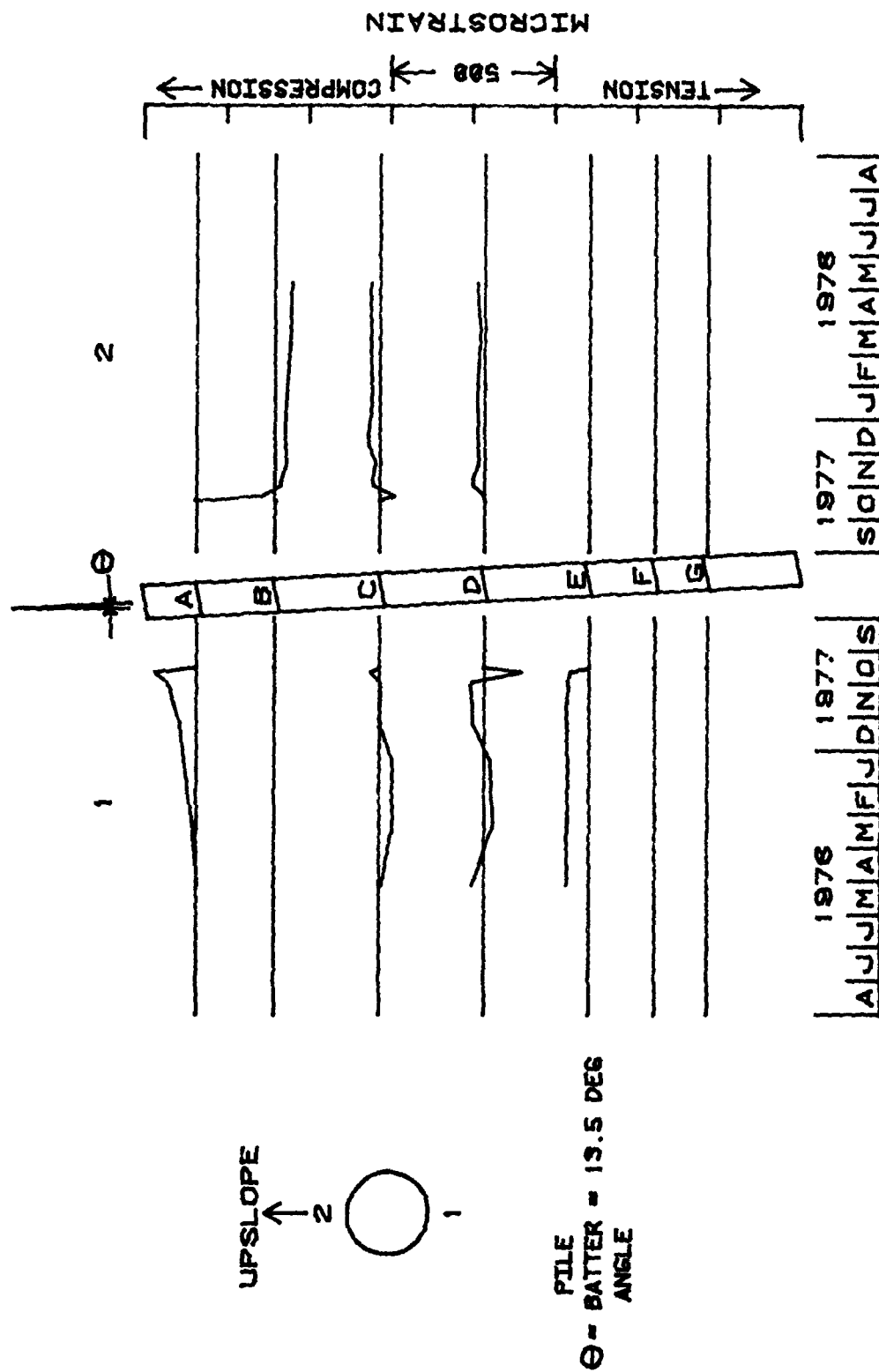


Figure 35. Strain-time histories along the length of pile 374, upslope and downslope

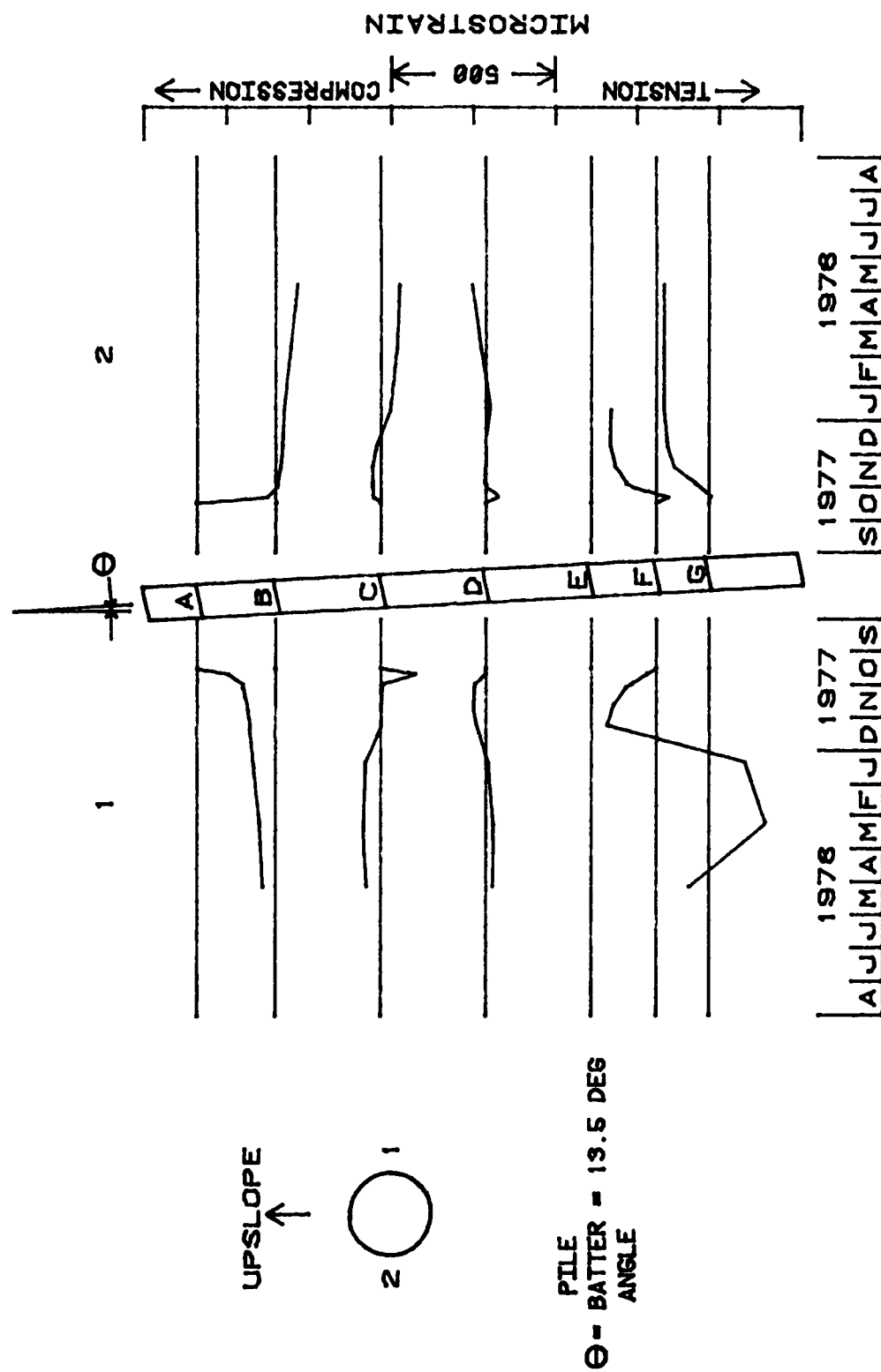


Figure 36. Strain-time histories along the length of pile 374, upstation and downstation

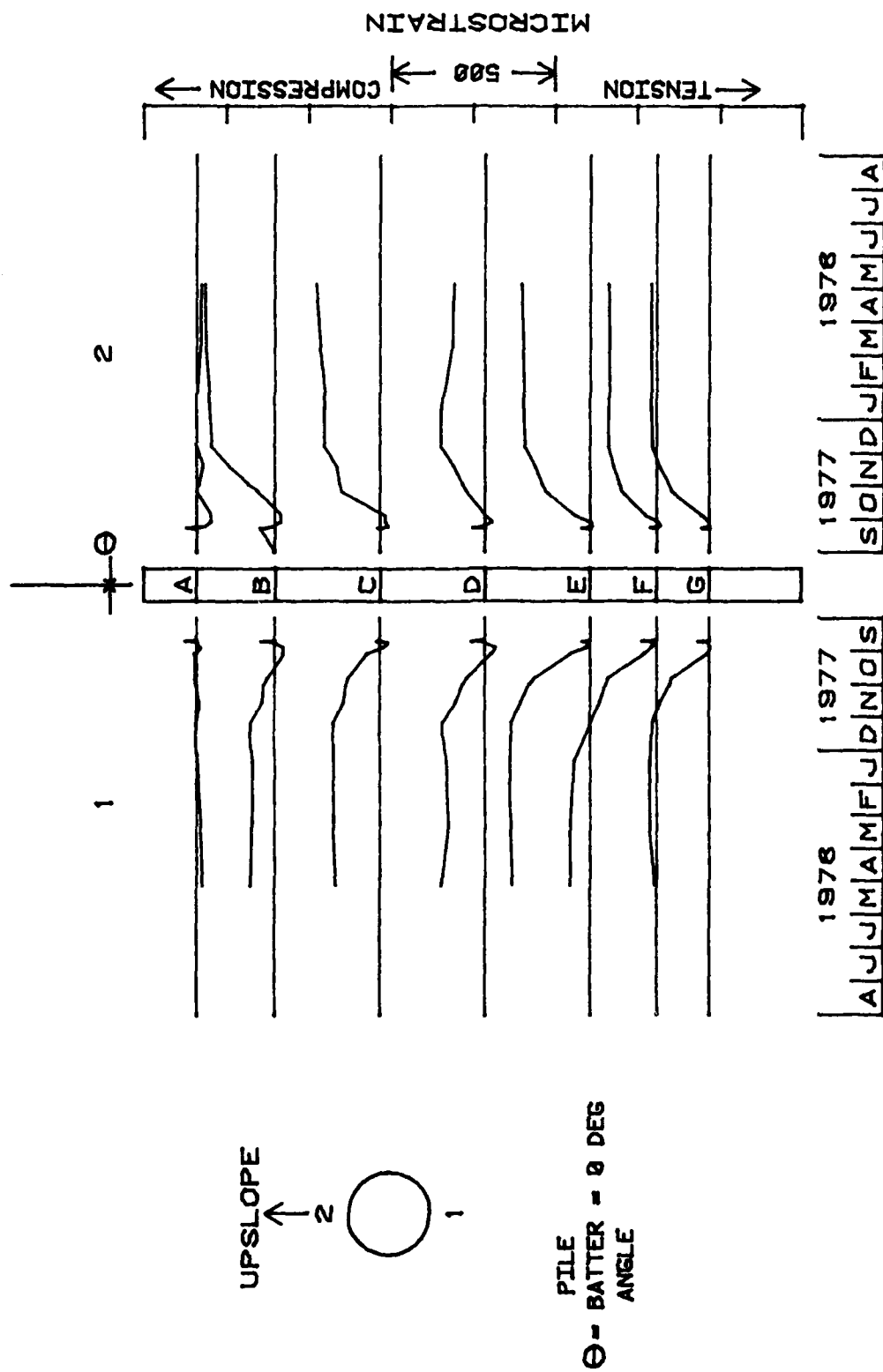


Figure 37. Strain-time histories along the length of pile 375, upslope and downslope

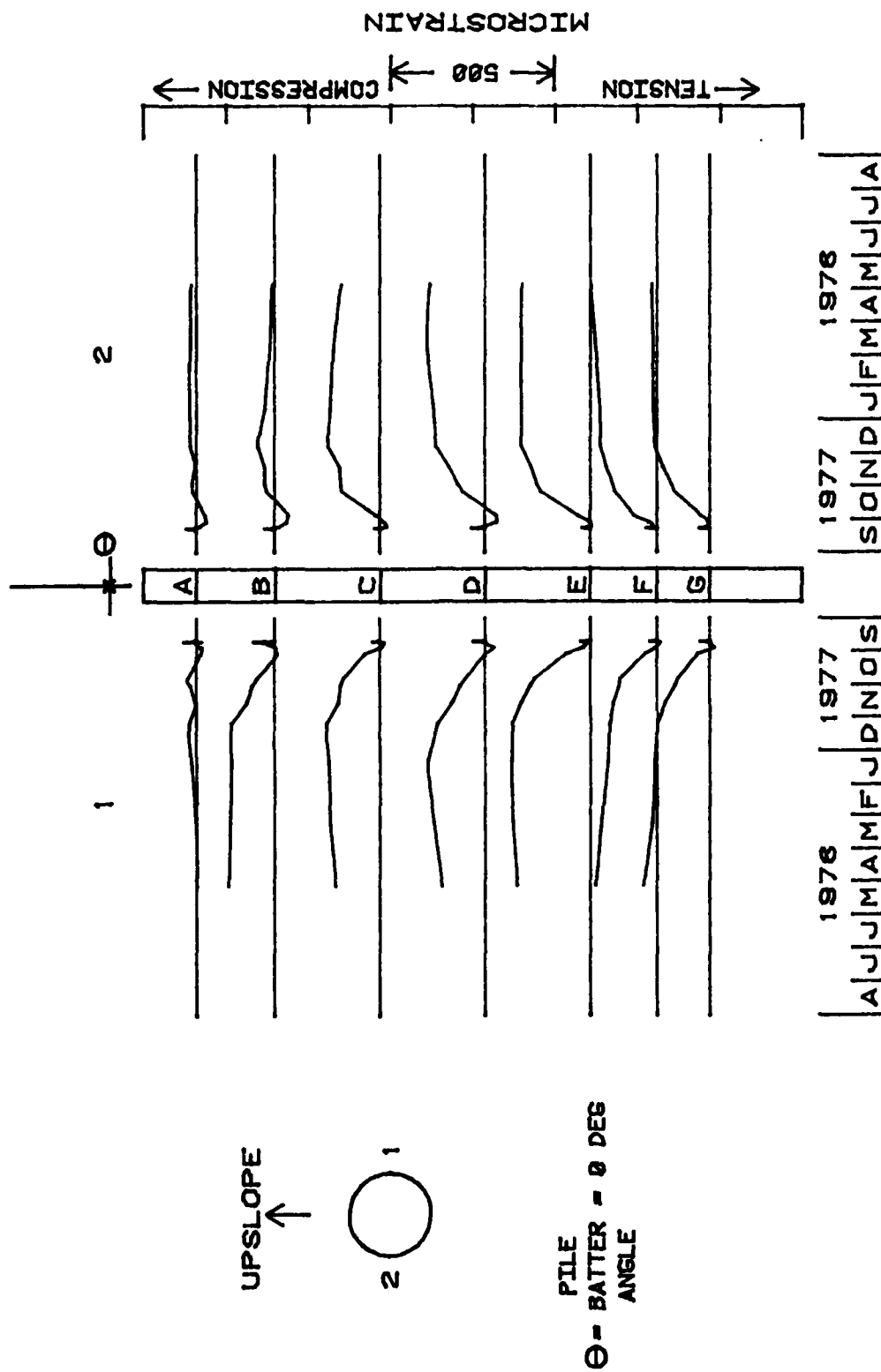


Figure 38. Strain-time histories along the length of pile 375, upstation and downstation

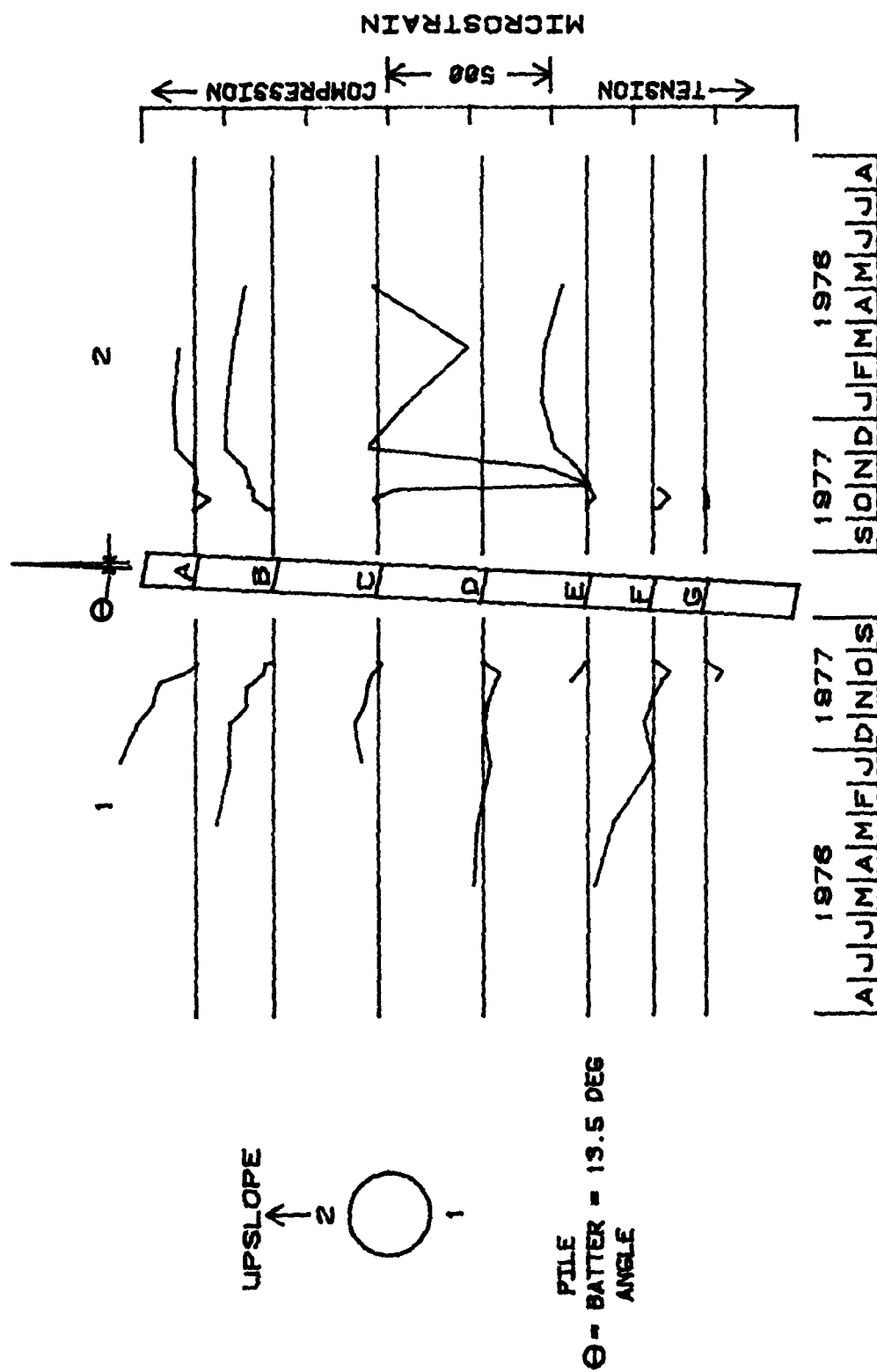


Figure 39. Strain-time histories along the length of pile 376, upslope and downslope

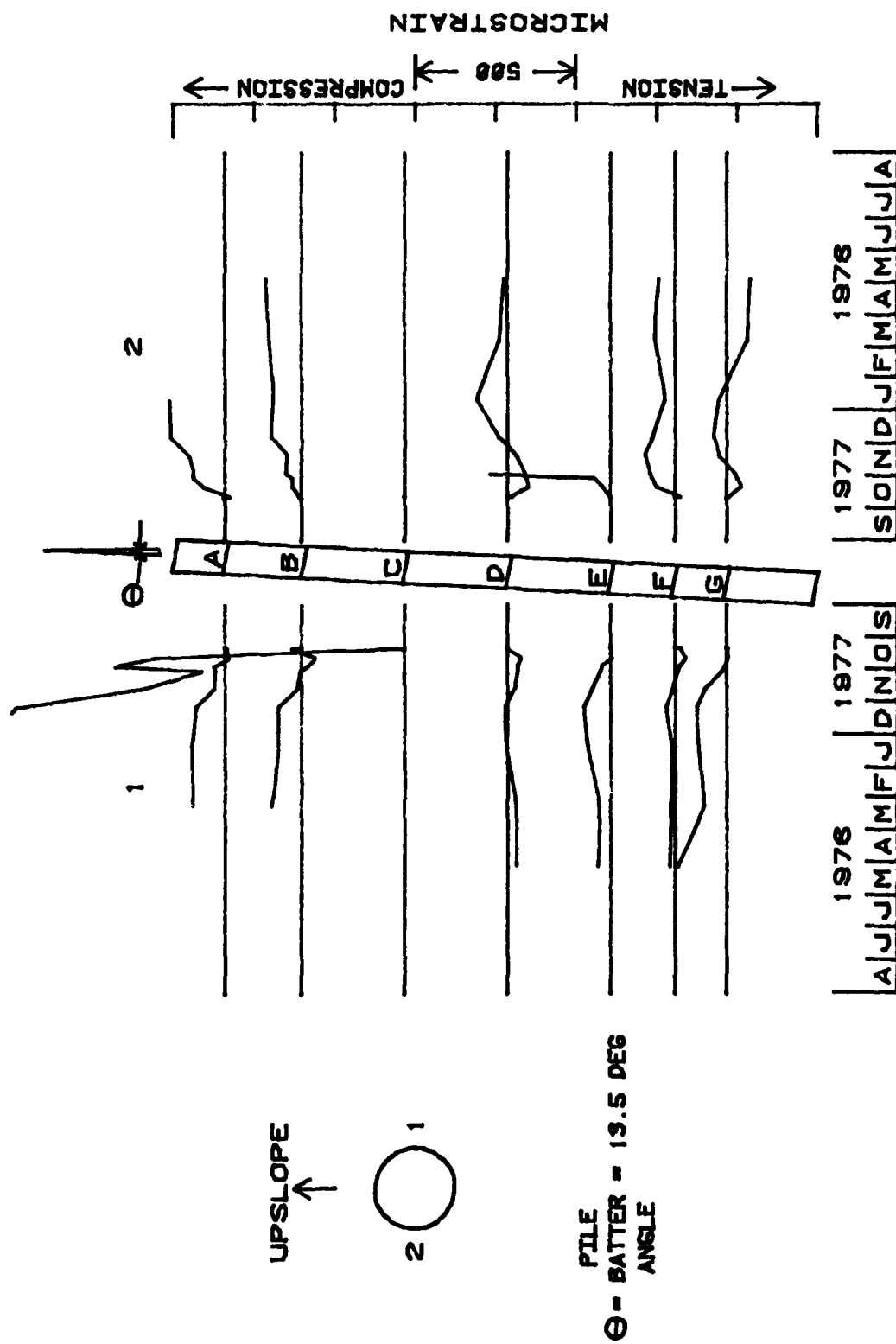


Figure 40. Strain-time histories along the length of pile 376, upstation and downstation

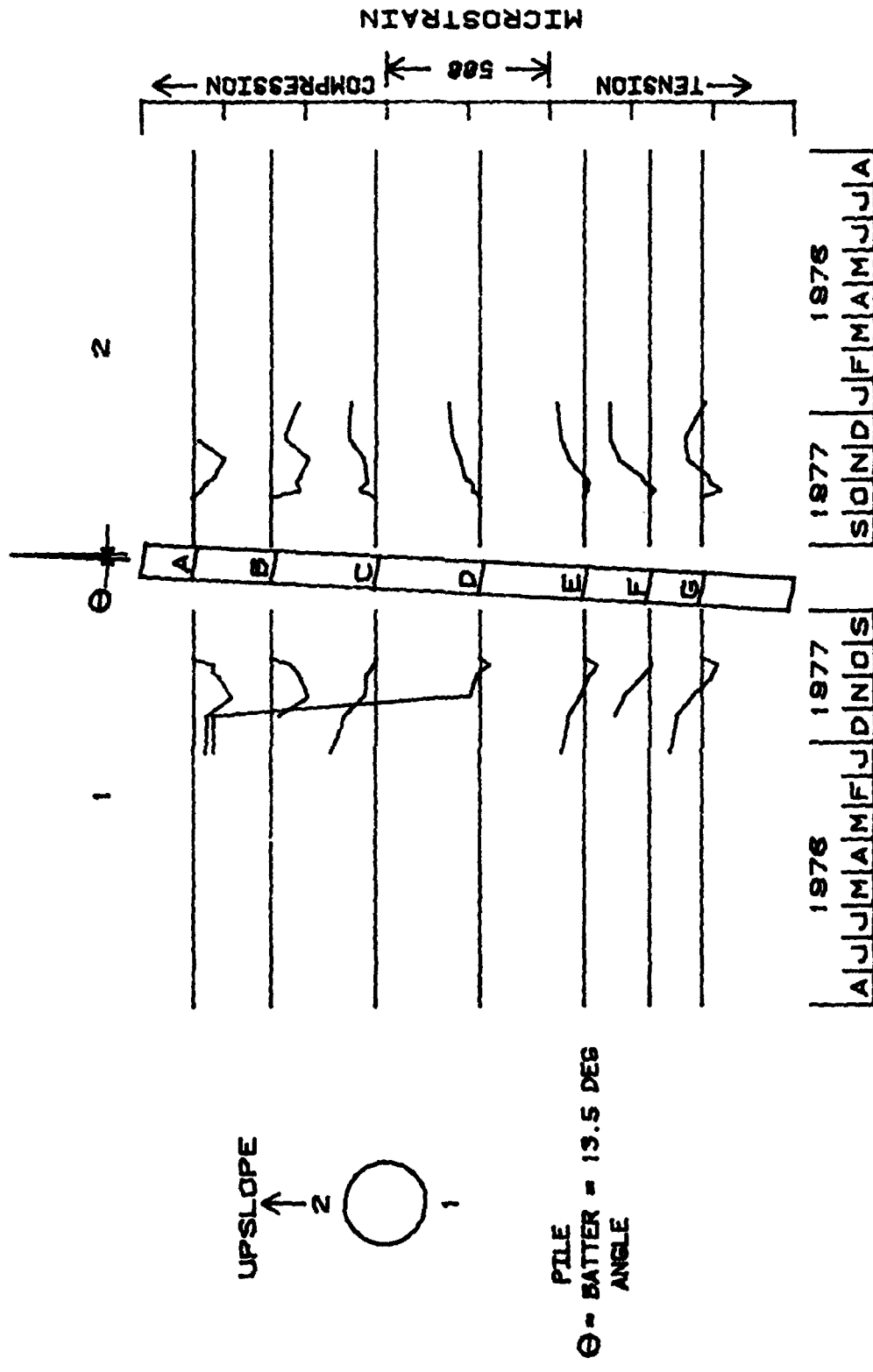


Figure 41. Strain-time histories along the length of pile 377, upslope and downslope

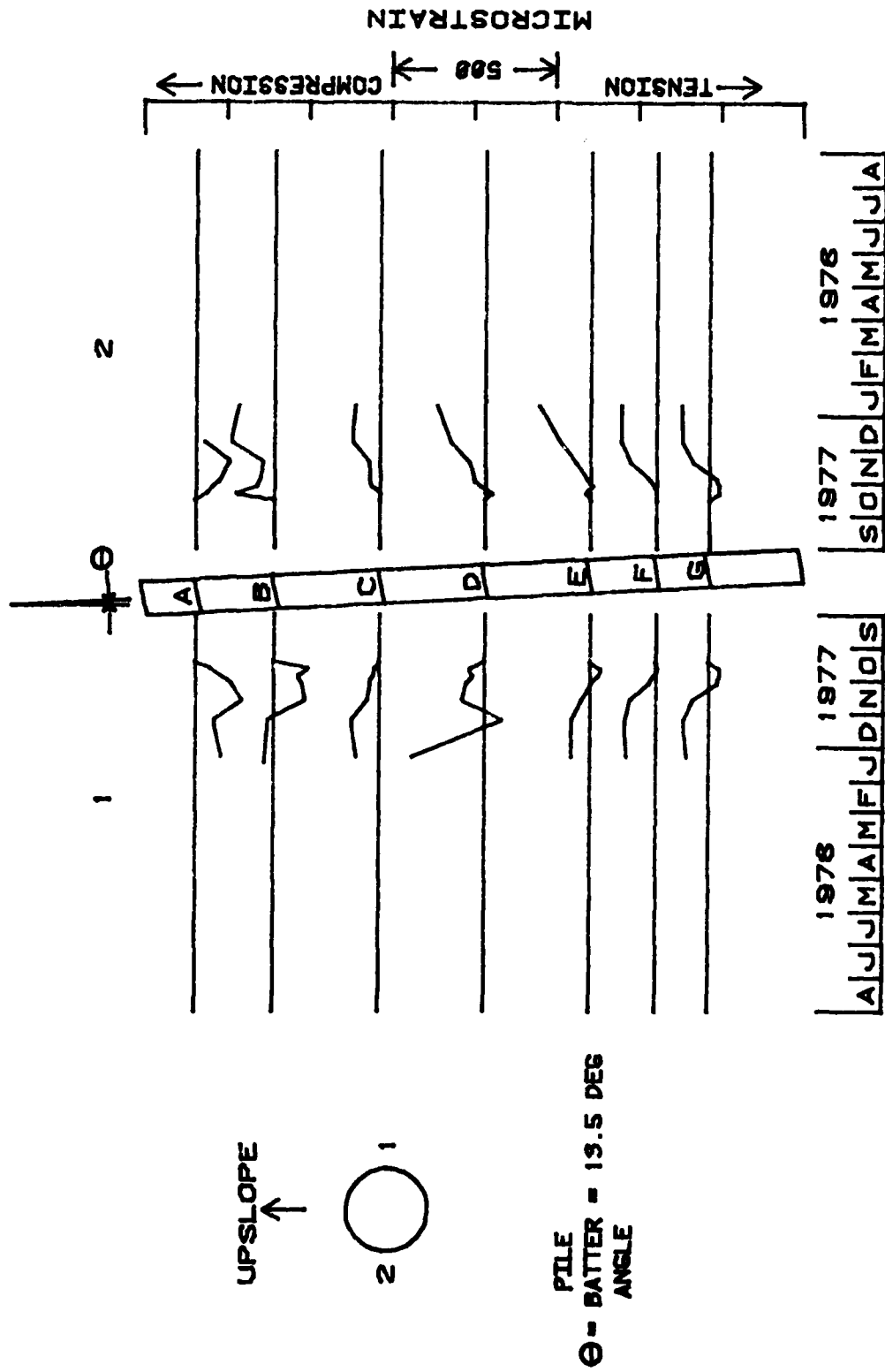


Figure 42. Strain-time histories along the length of pile 377, upstation and downstation

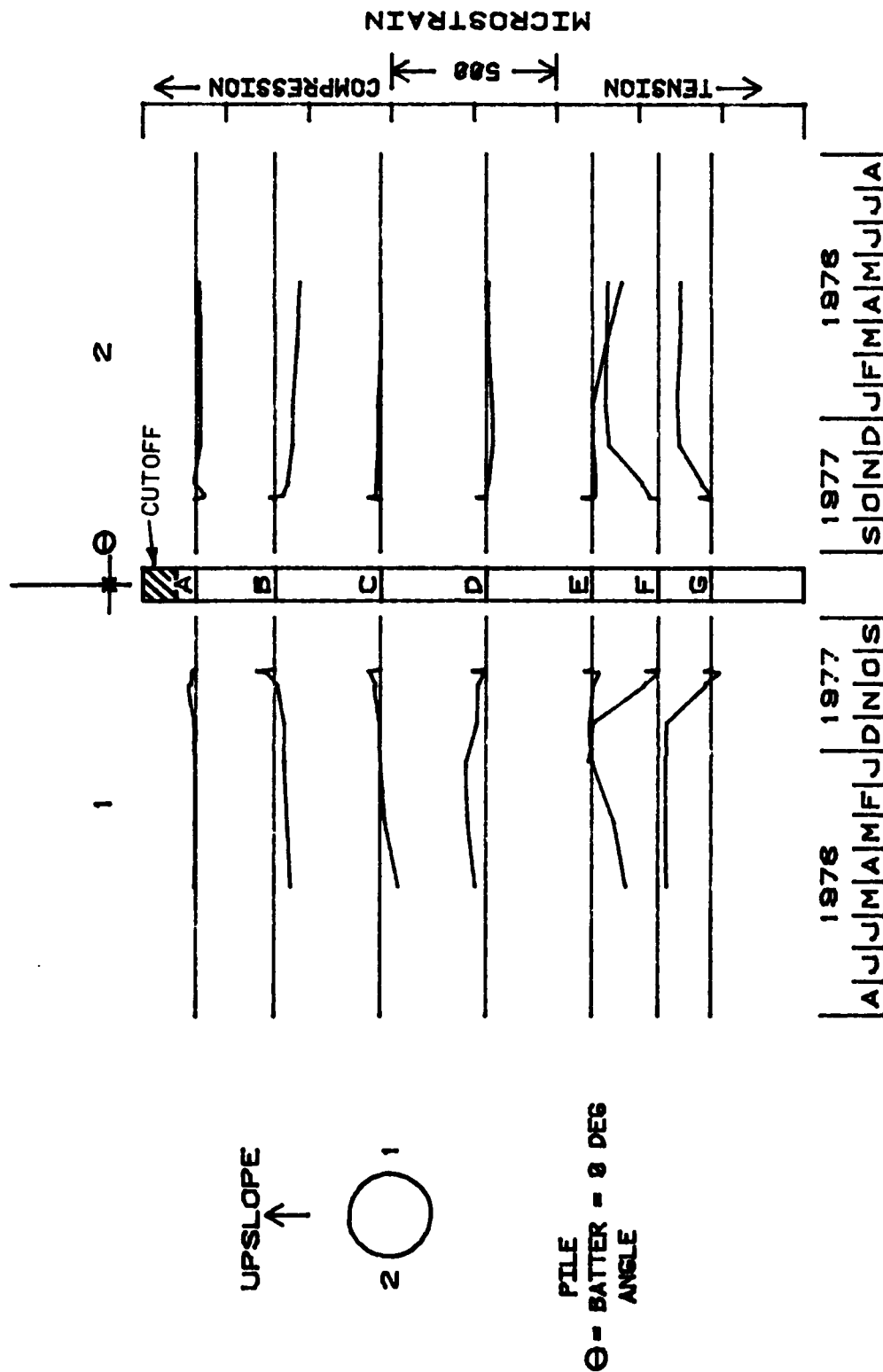


Figure 44. Strain-time histories along the length of pile 378, upstation and downstation

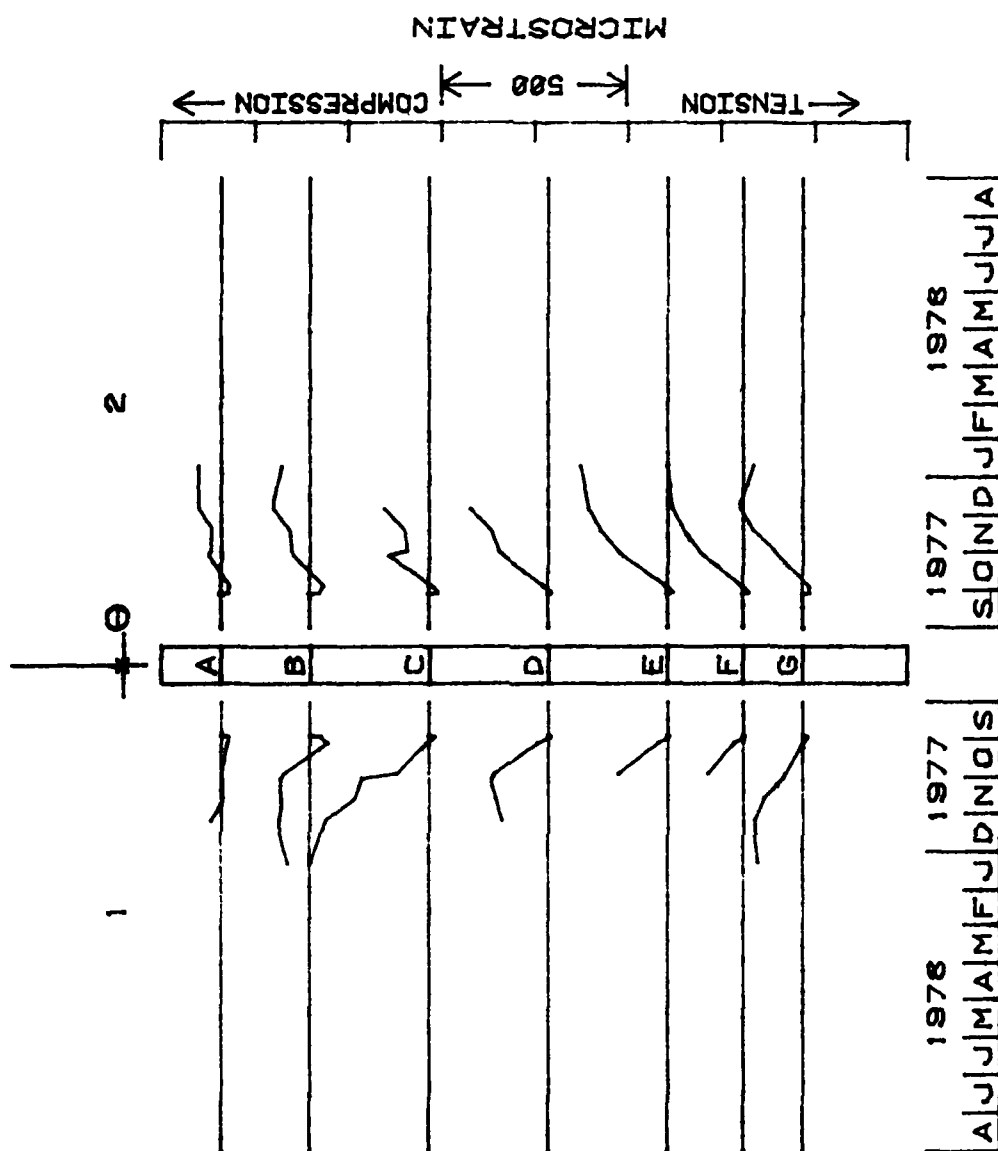
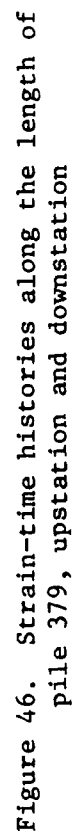
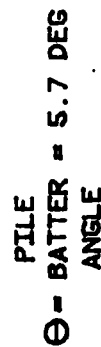


Figure 45. Strain-time histories along the length of pile 379, upslope and downslope



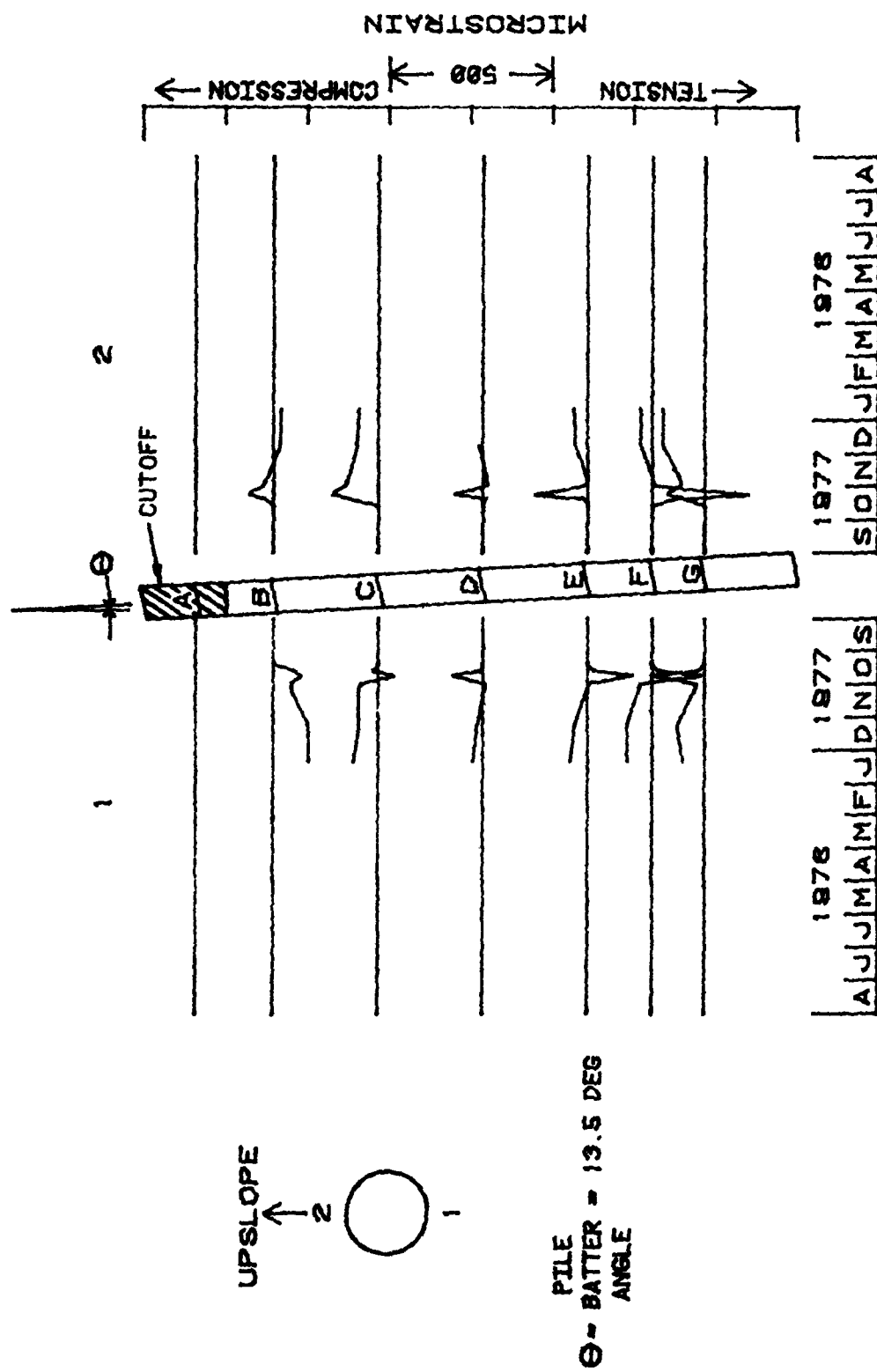


Figure 47. Strain-time histories along the length of pile 380, upslope and downslope

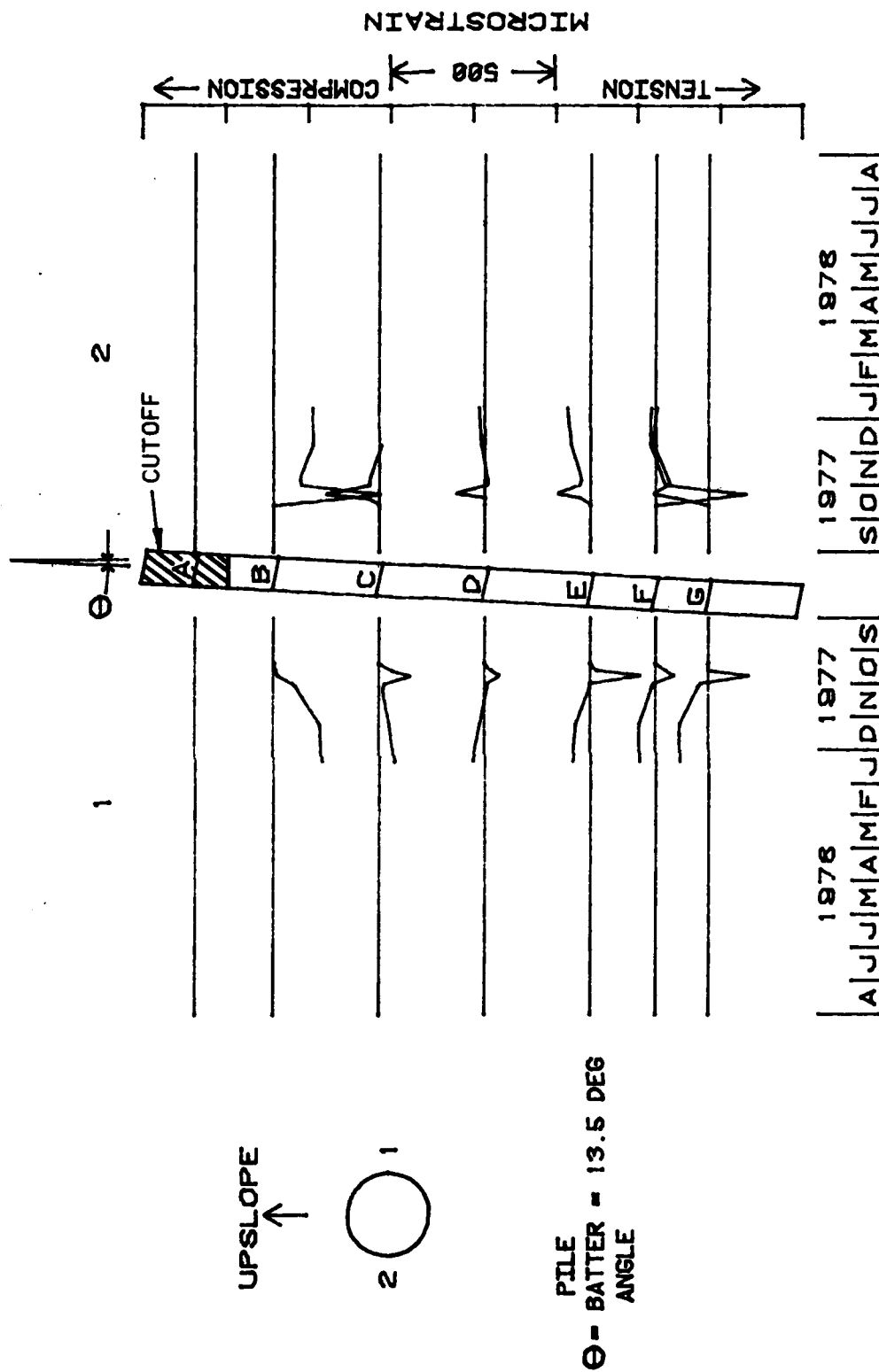
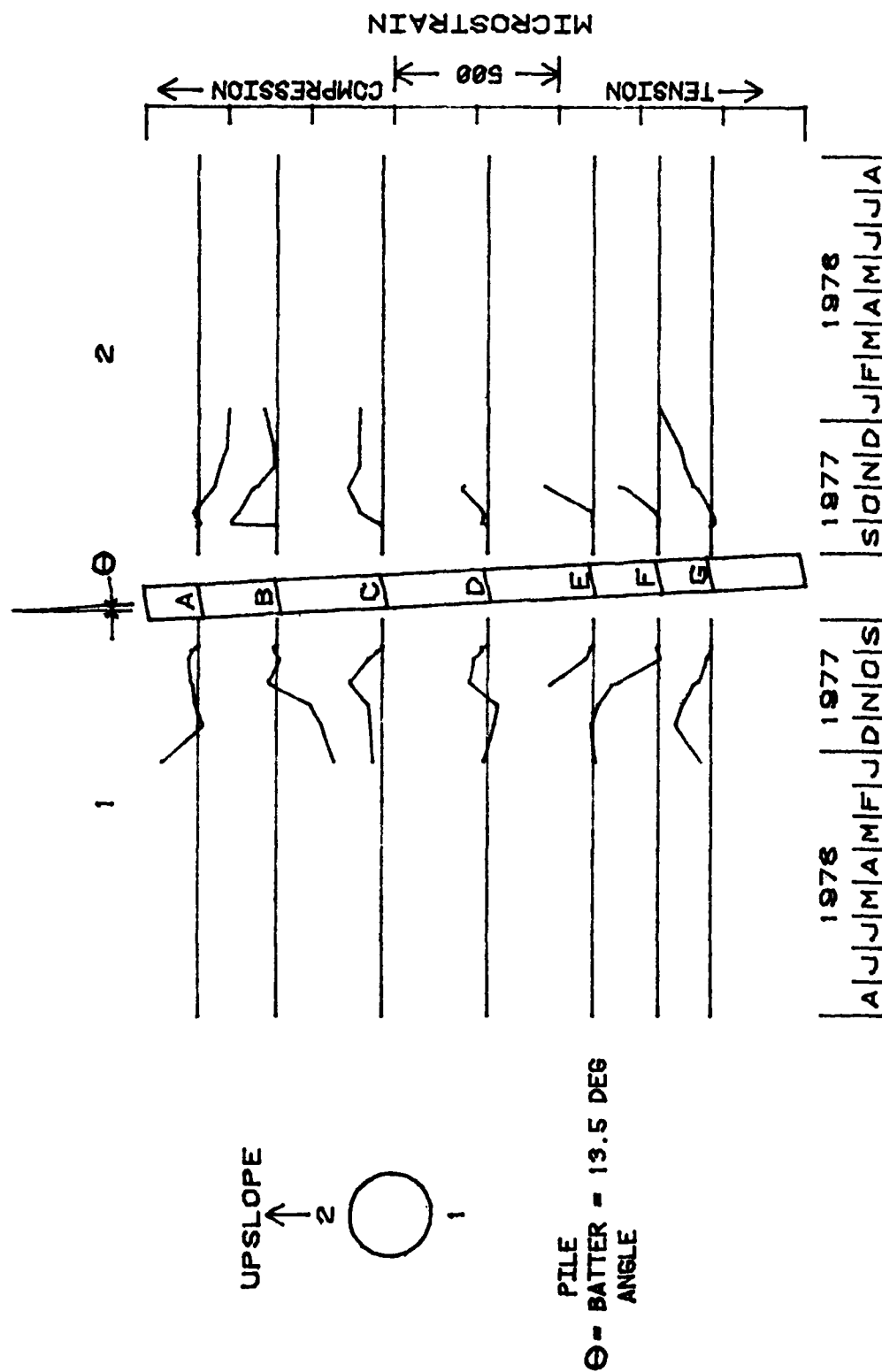


Figure 48. Strain-time histories along the length of pile 380, upstation and downstation



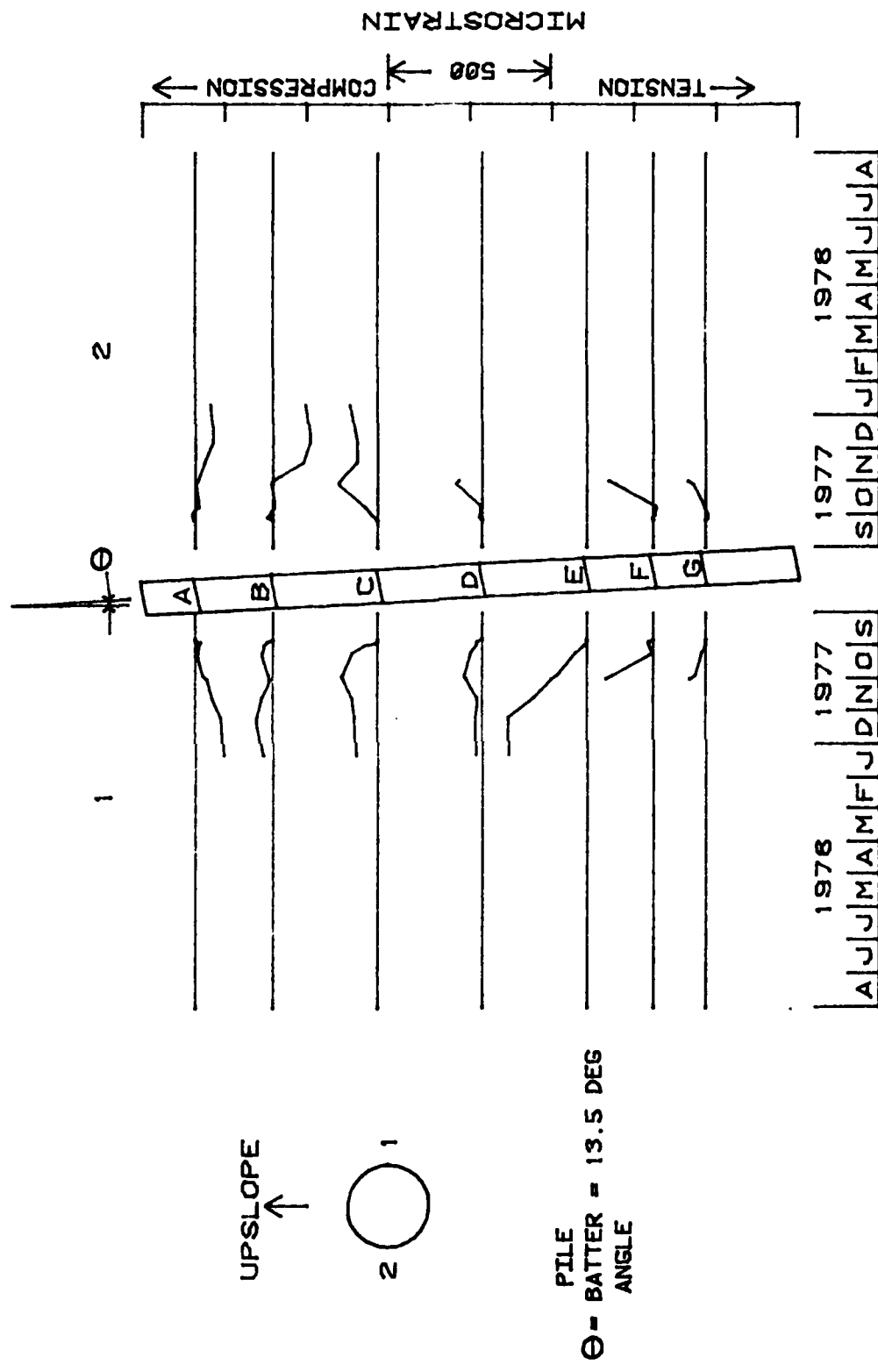


Figure 50. Strain-time histories along the length of pile 381, upstation and downstation

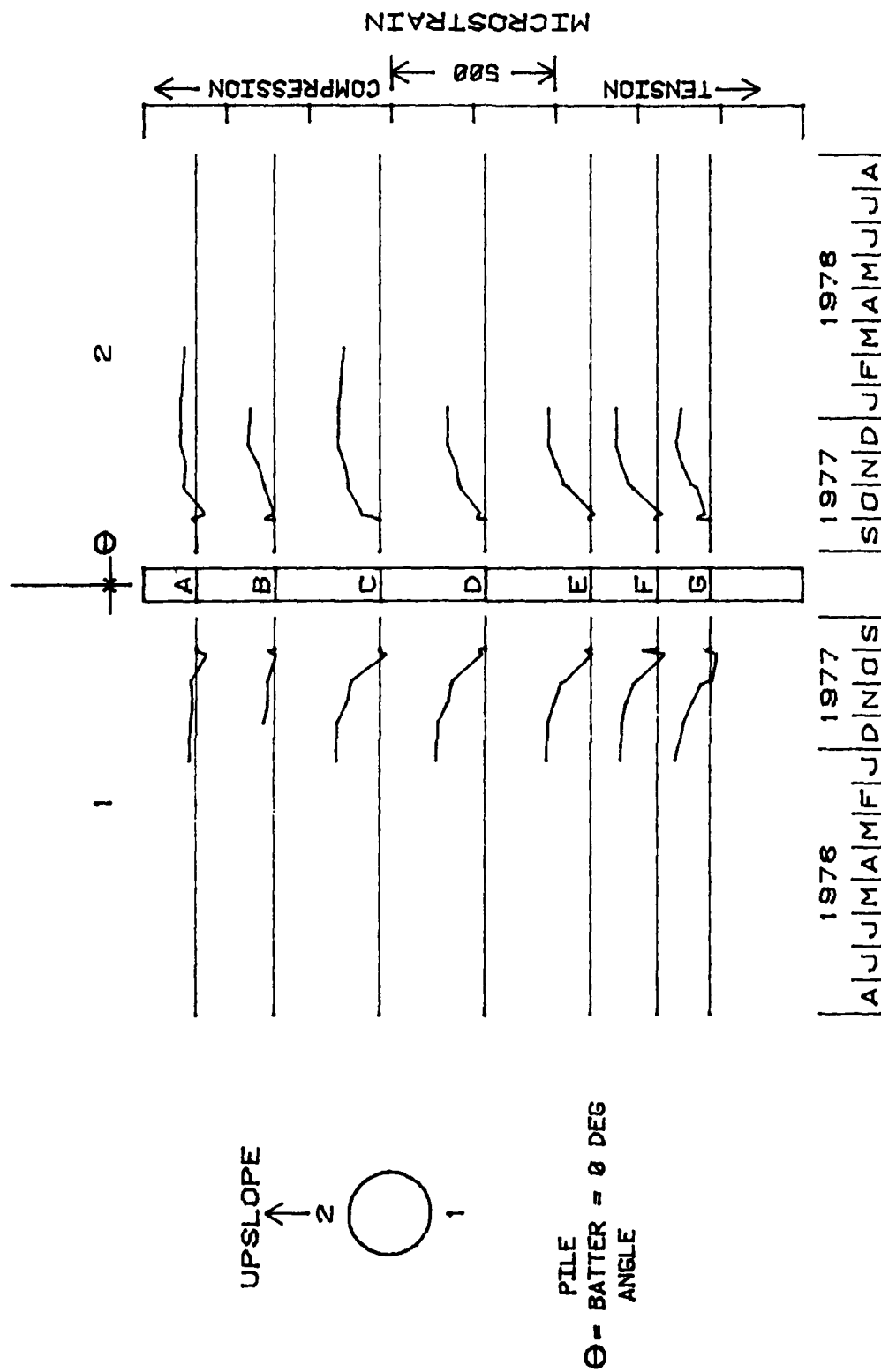


Figure 51. Strain-time histories along the length of pile 382, upslope and downslope

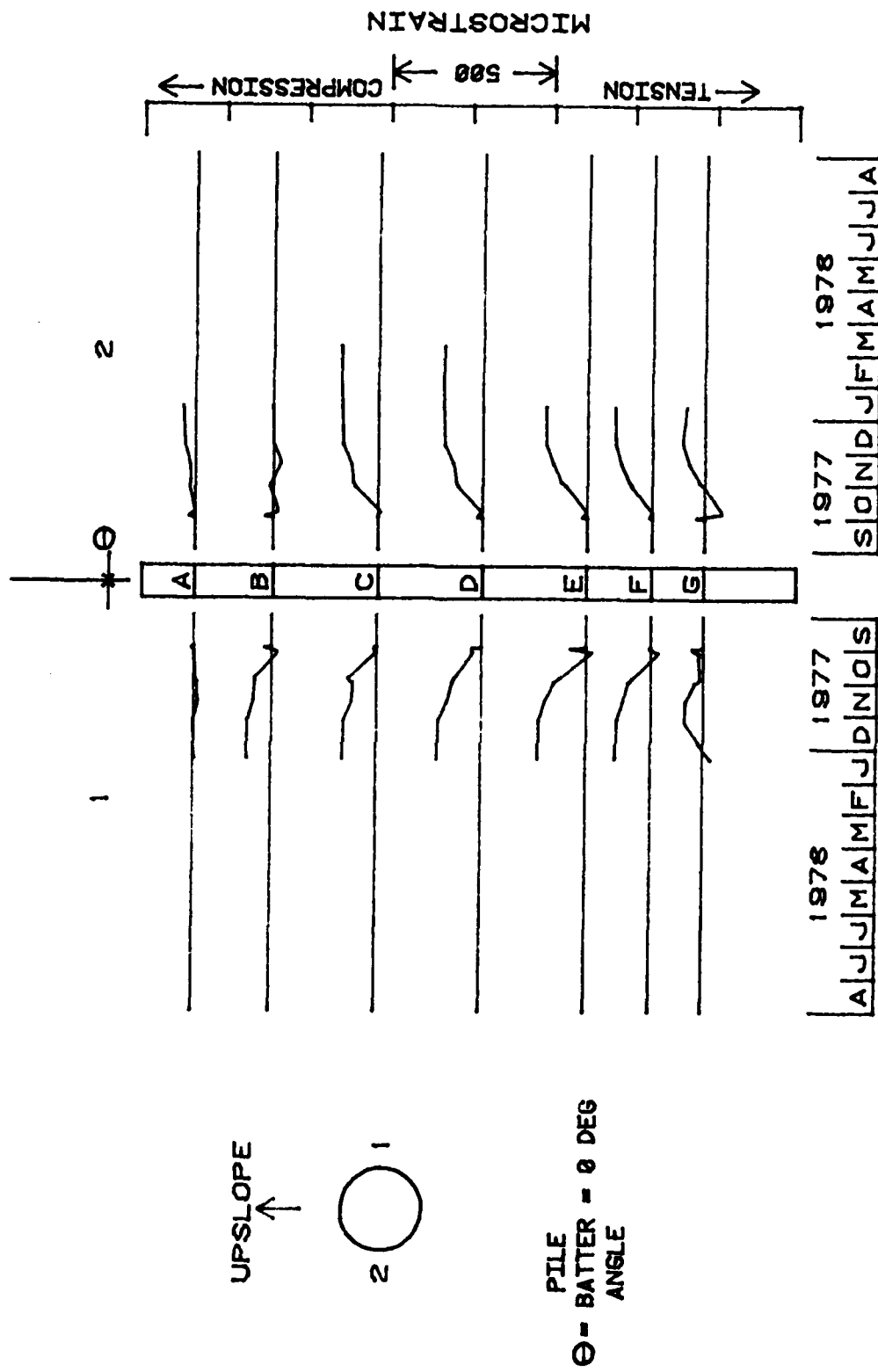


Figure 52. Strain-time histories along the length of pile 382, upstation and downstation

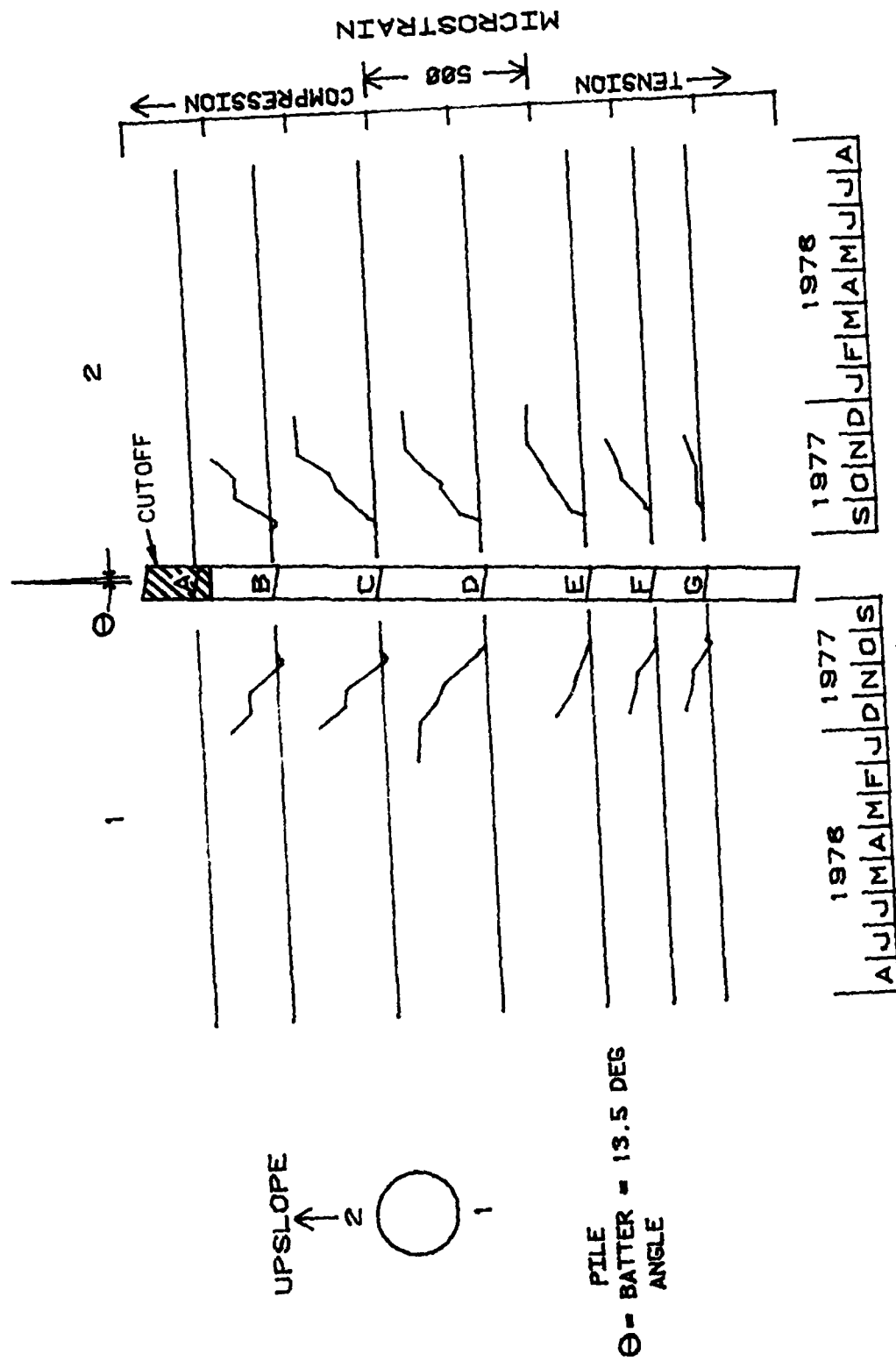


Figure 53. Strain-time histories along the length of pile 383, upslope and downslope

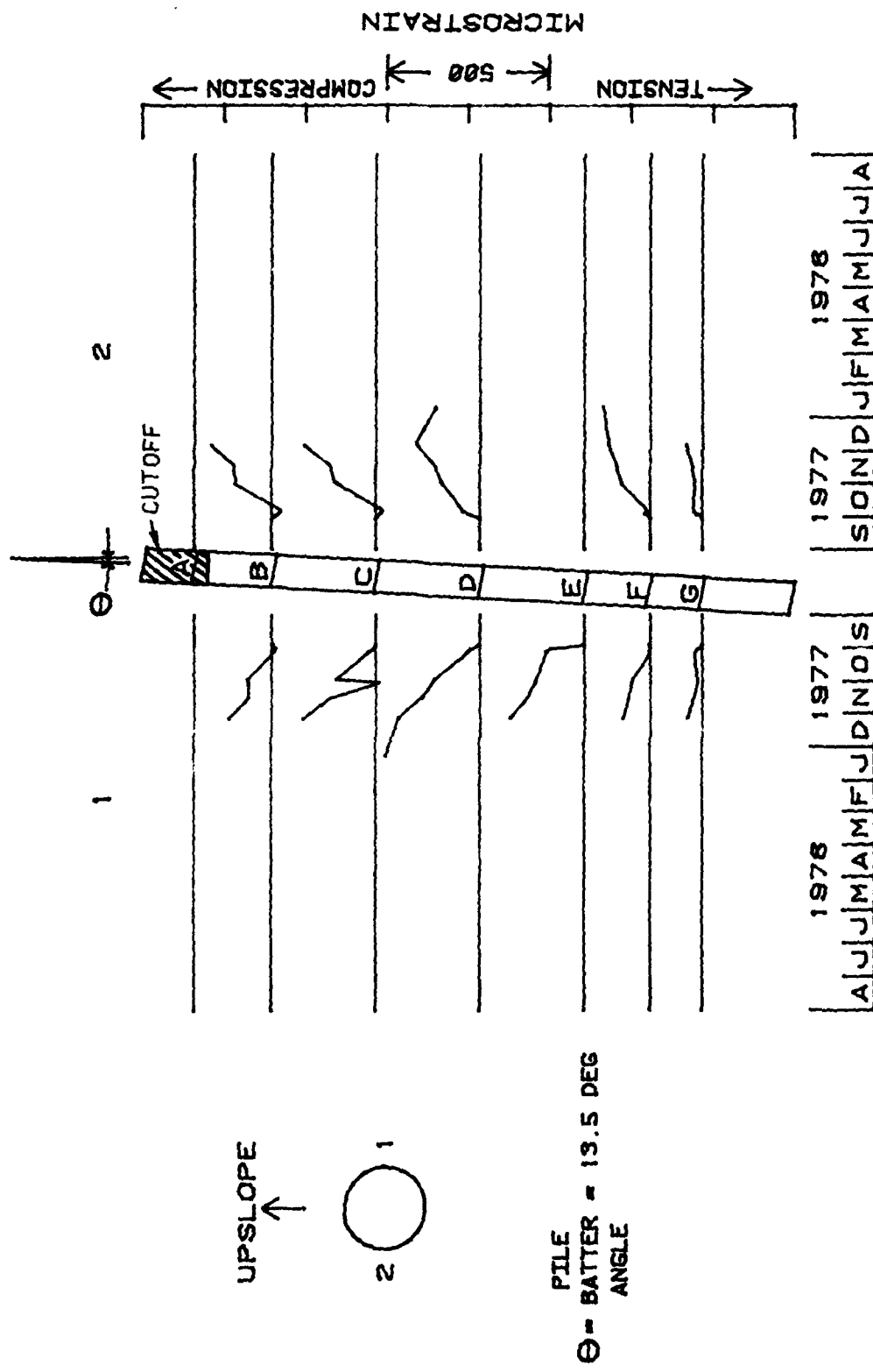



Figure 54. Strain-time histories along the length of pile 383, upstation and downstation

U P S L O P E



PILE
 Θ - BATTER = 13.5 DEG
 ANGLE

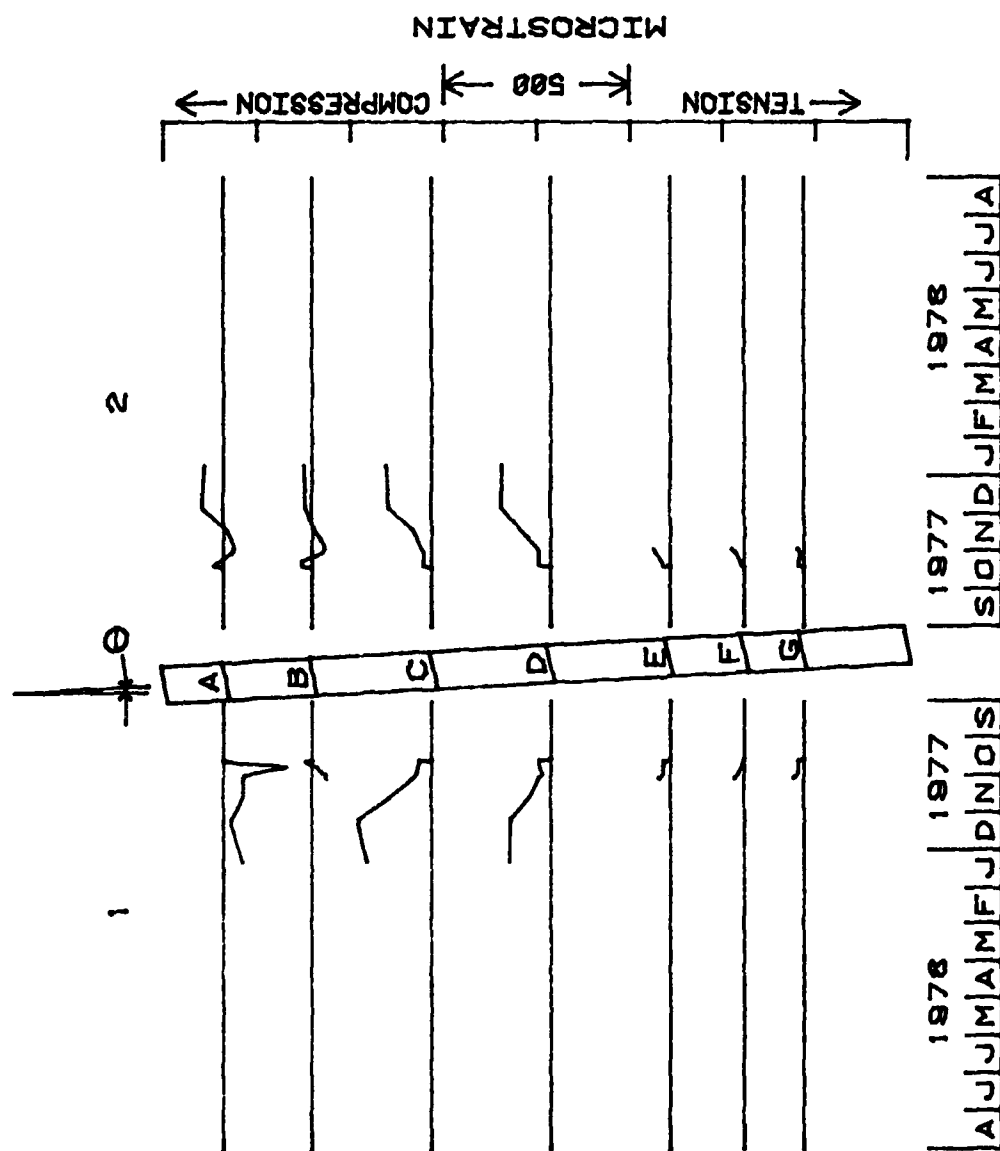
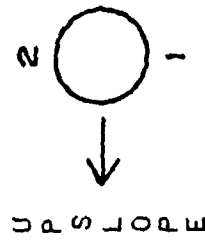


Figure 55. Strain-time histories along the length of pile 384, upslope and downslope



PILE
 Θ - BATTER = 13.5 DEG
 ANGLE

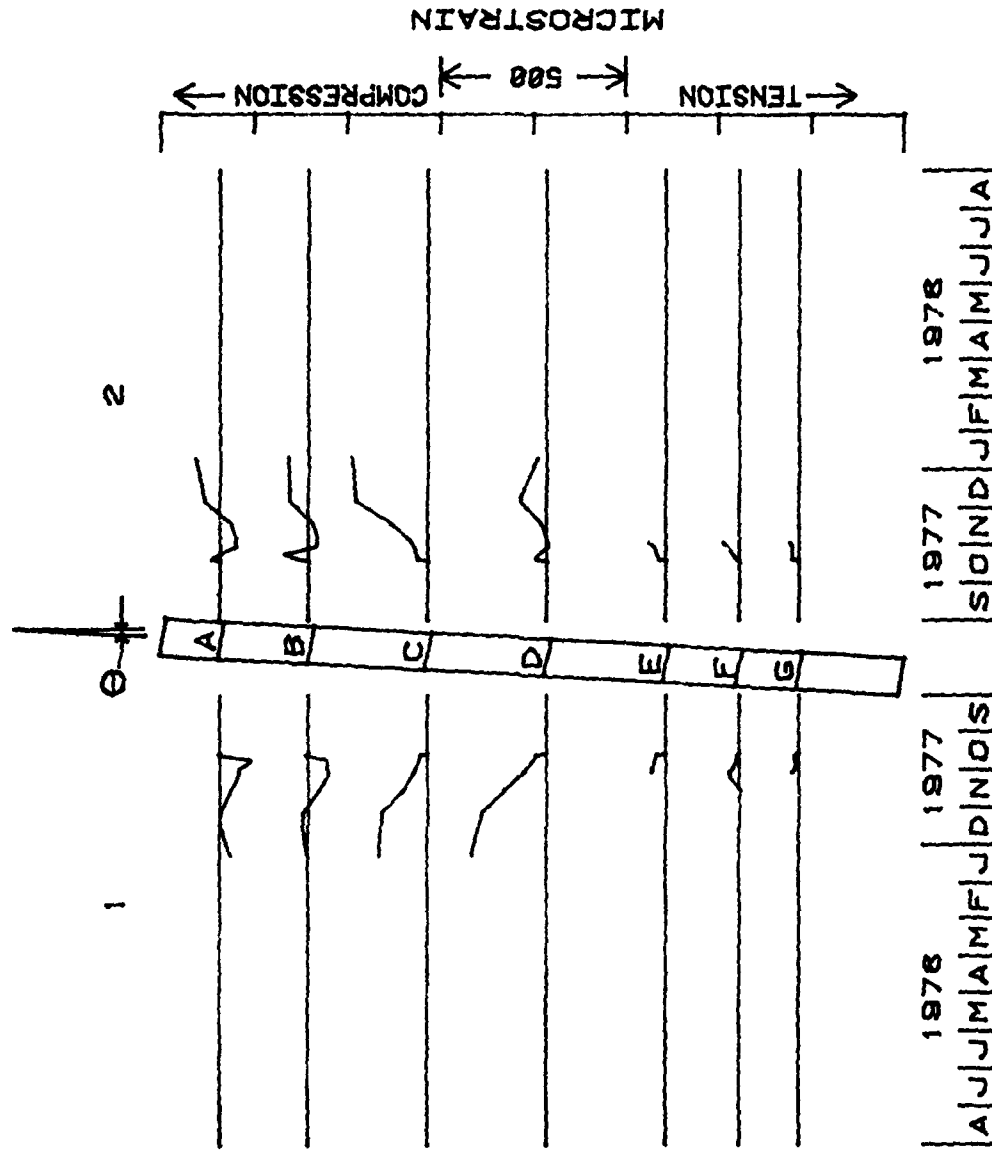


Figure 56. Strain-time histories along the length of pile 384, upstation and downstation

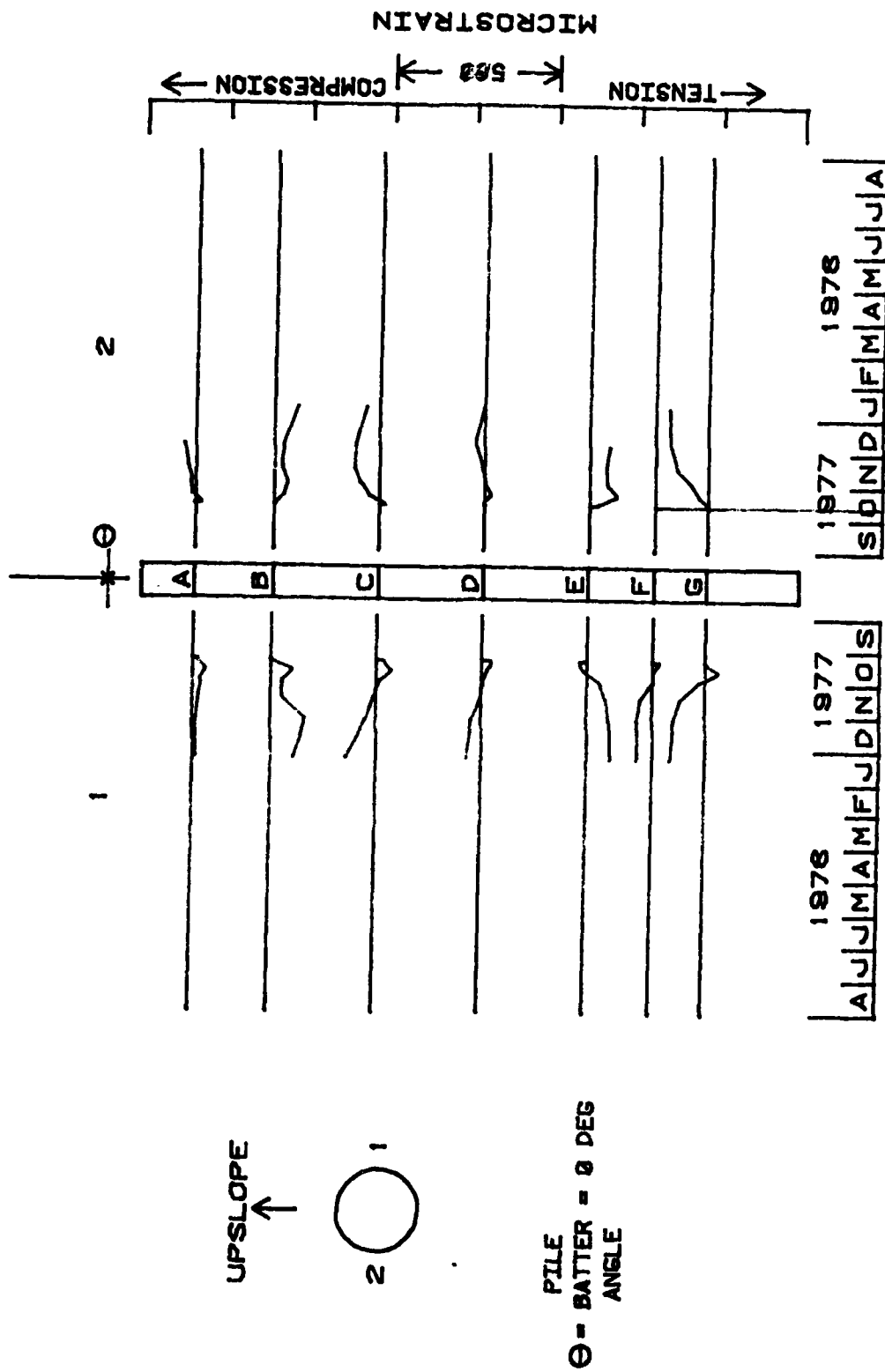


Figure 58. Strain-time histories along the length of pile 385, upstation and downstation

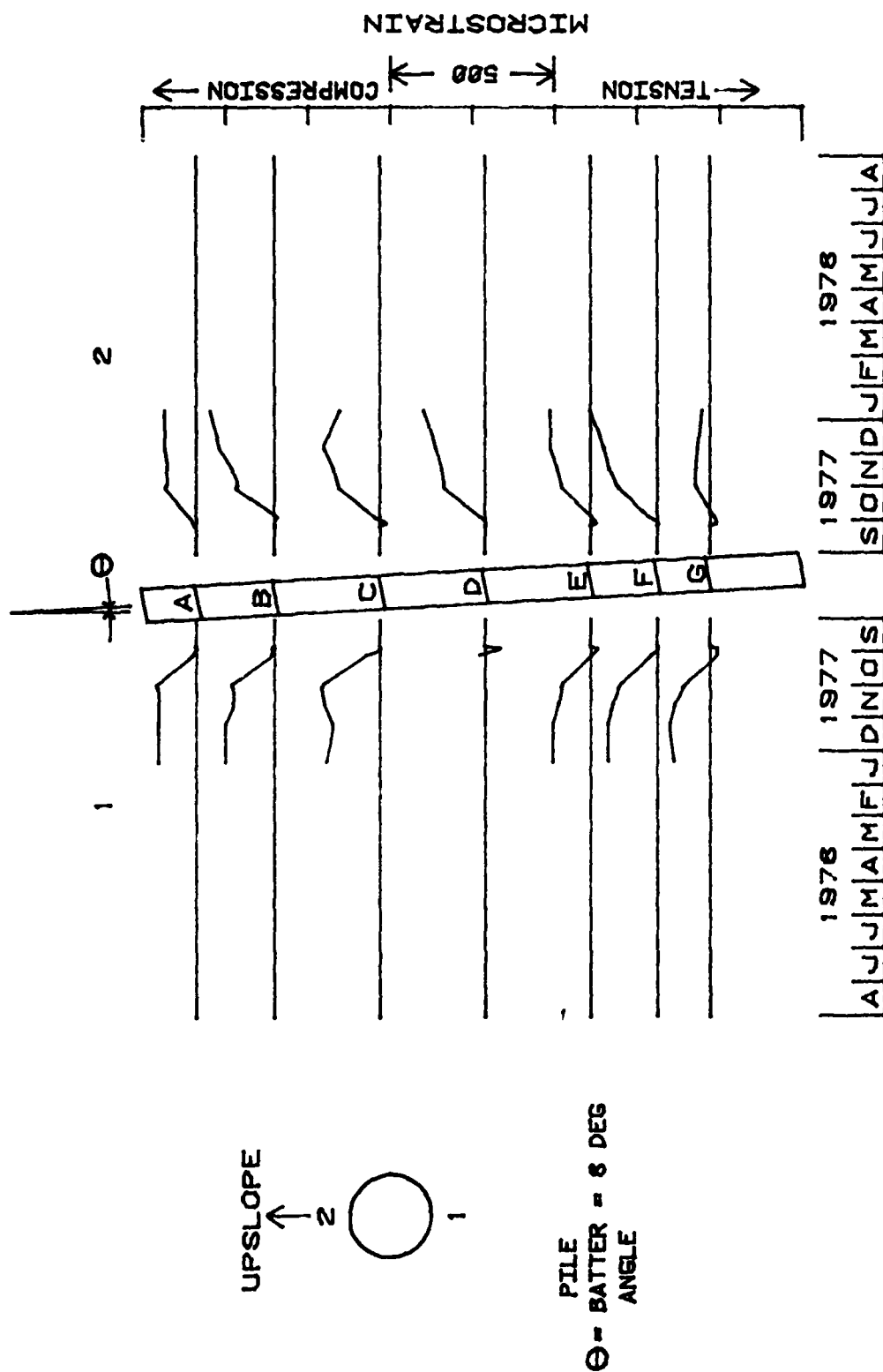


Figure 59. Strain-time histories along the length of pile 386, upslope and downslope

UPSLOPE



PILE
BATTER = 0 DEG
ANGLE

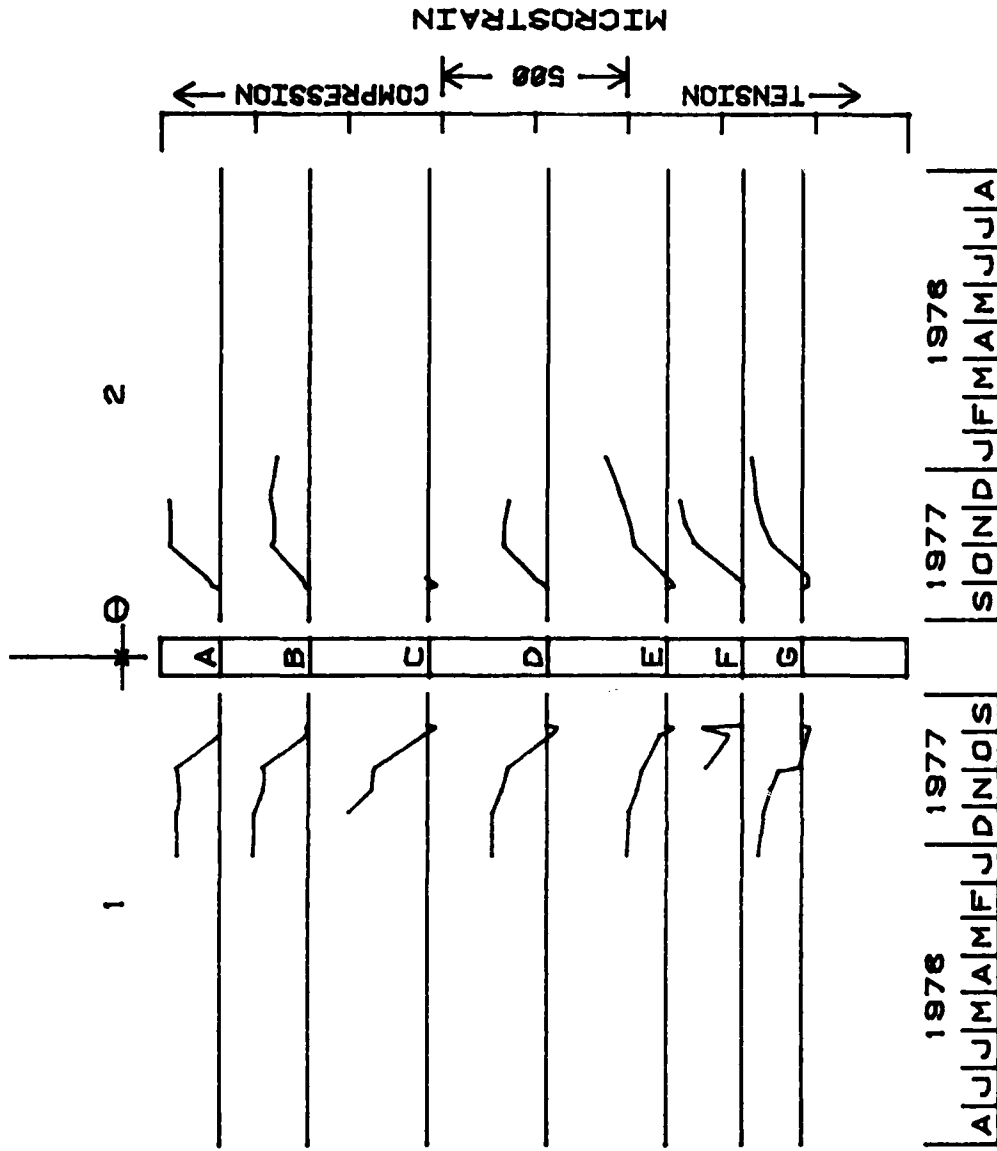
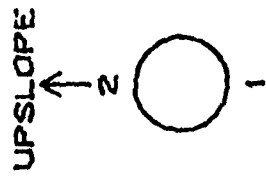


Figure 60. Strain-time histories along the length of pile 386, upstation and downstation



PILE
BATTER = 8 DEG
ANGLE

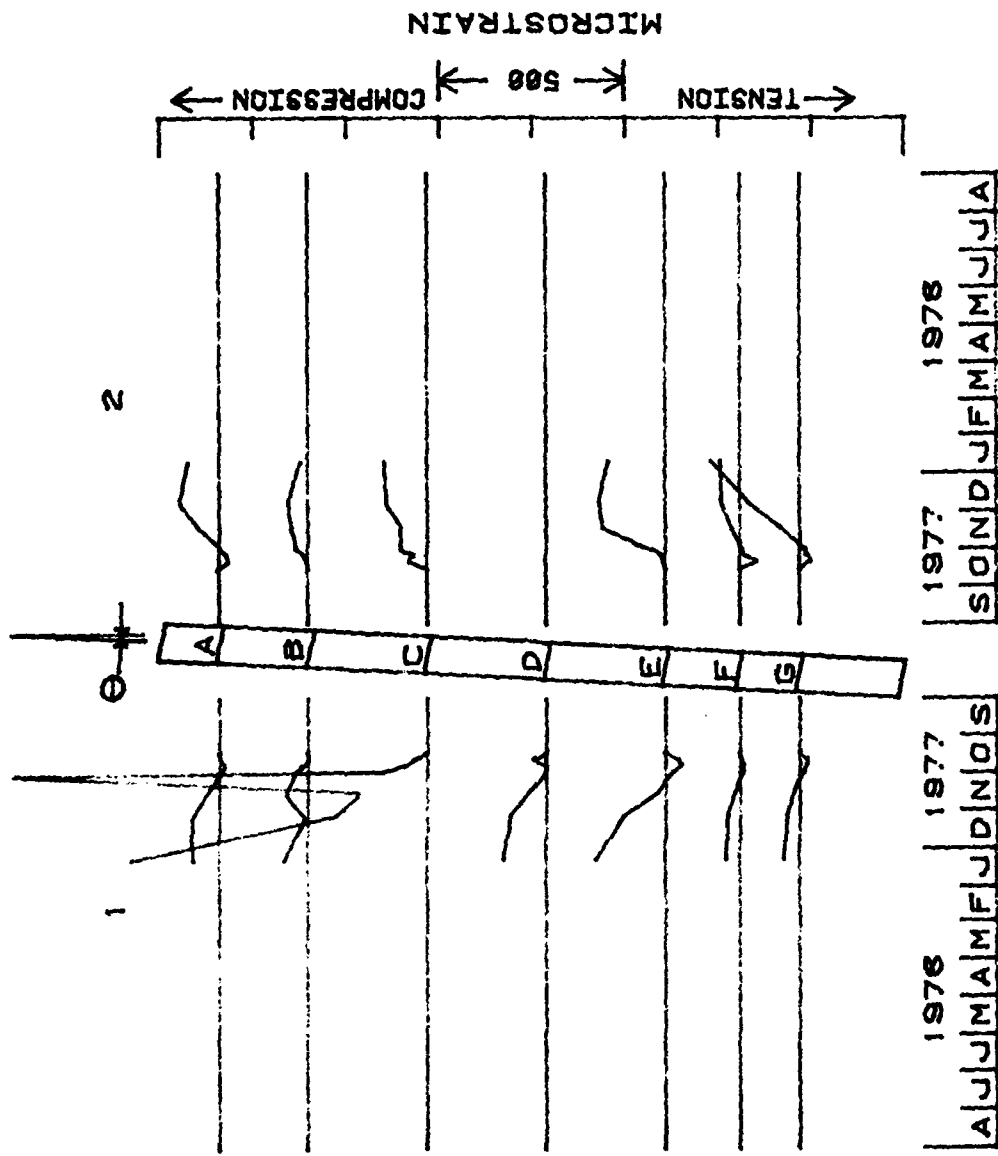


Figure 61. Strain-time histories along the length of pile 392, upslope and downslope

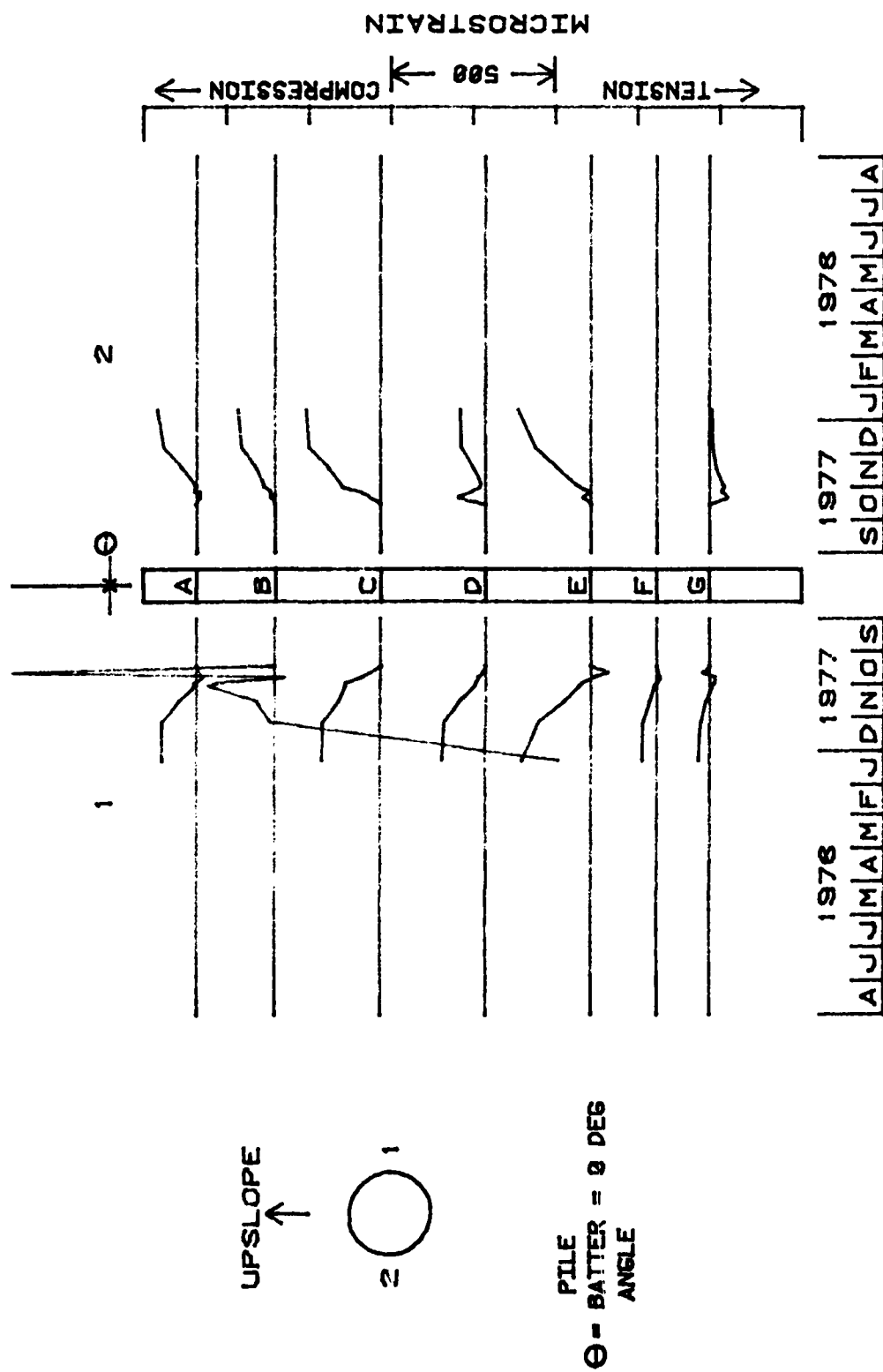


Figure 62. Strain-time histories along the length of pile 392, upstation and downstation

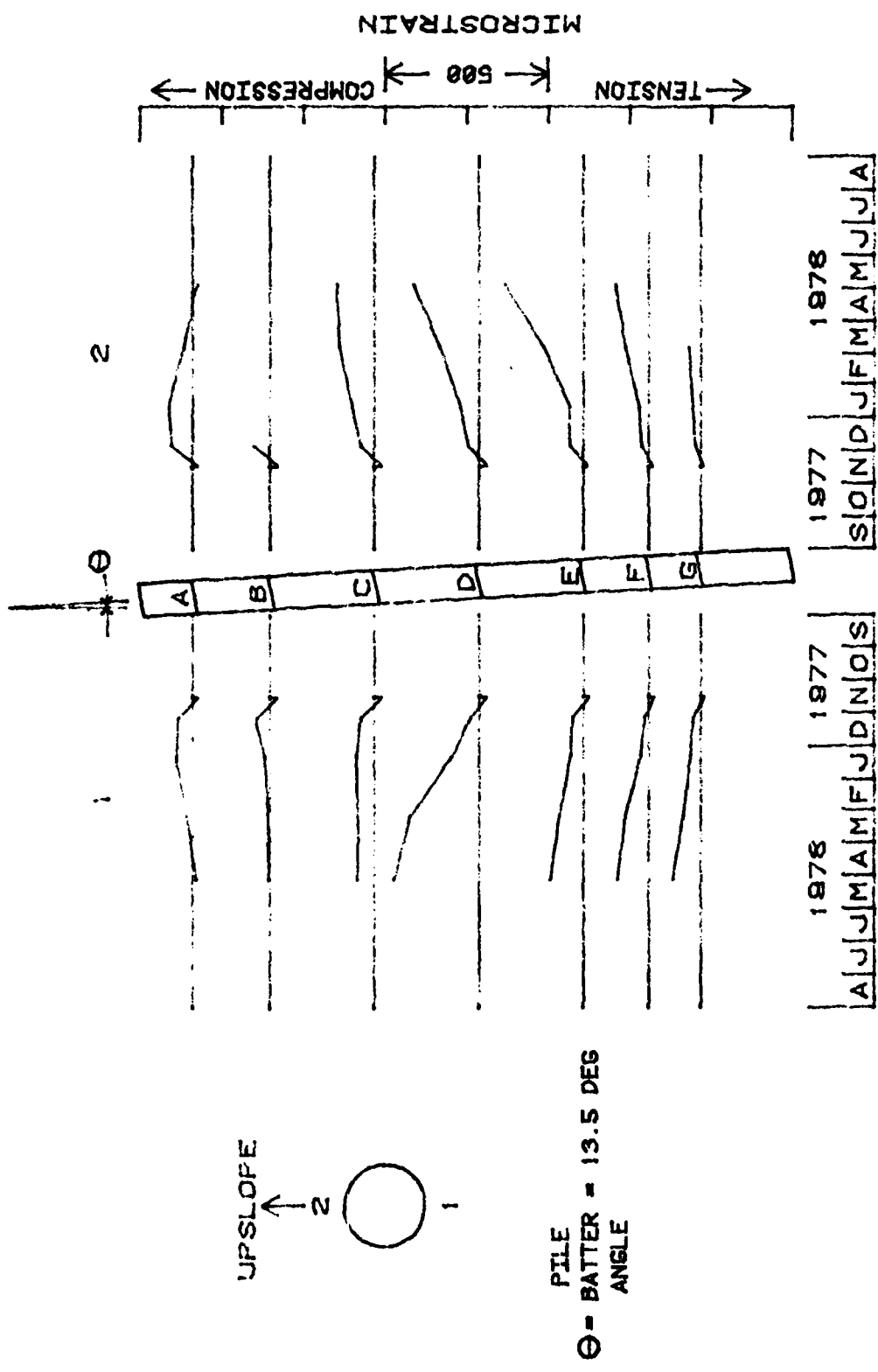


Figure 63. Strain-time histories along the length of pile 404, upslope and downslope

AD-A142 689

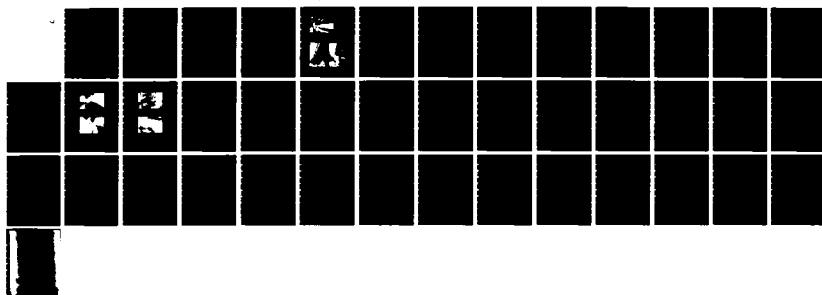
STABILIZATION OF MOVING LAND MASSES BY CAST-IN-PLACE
PILES(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION
VICKSBURG MS GEOTECHNICAL LAB J B PALMERTON MAY 84
WES/MP/GL-84-4

2/2

UNCLASSIFIED

F/G 8/7

NL



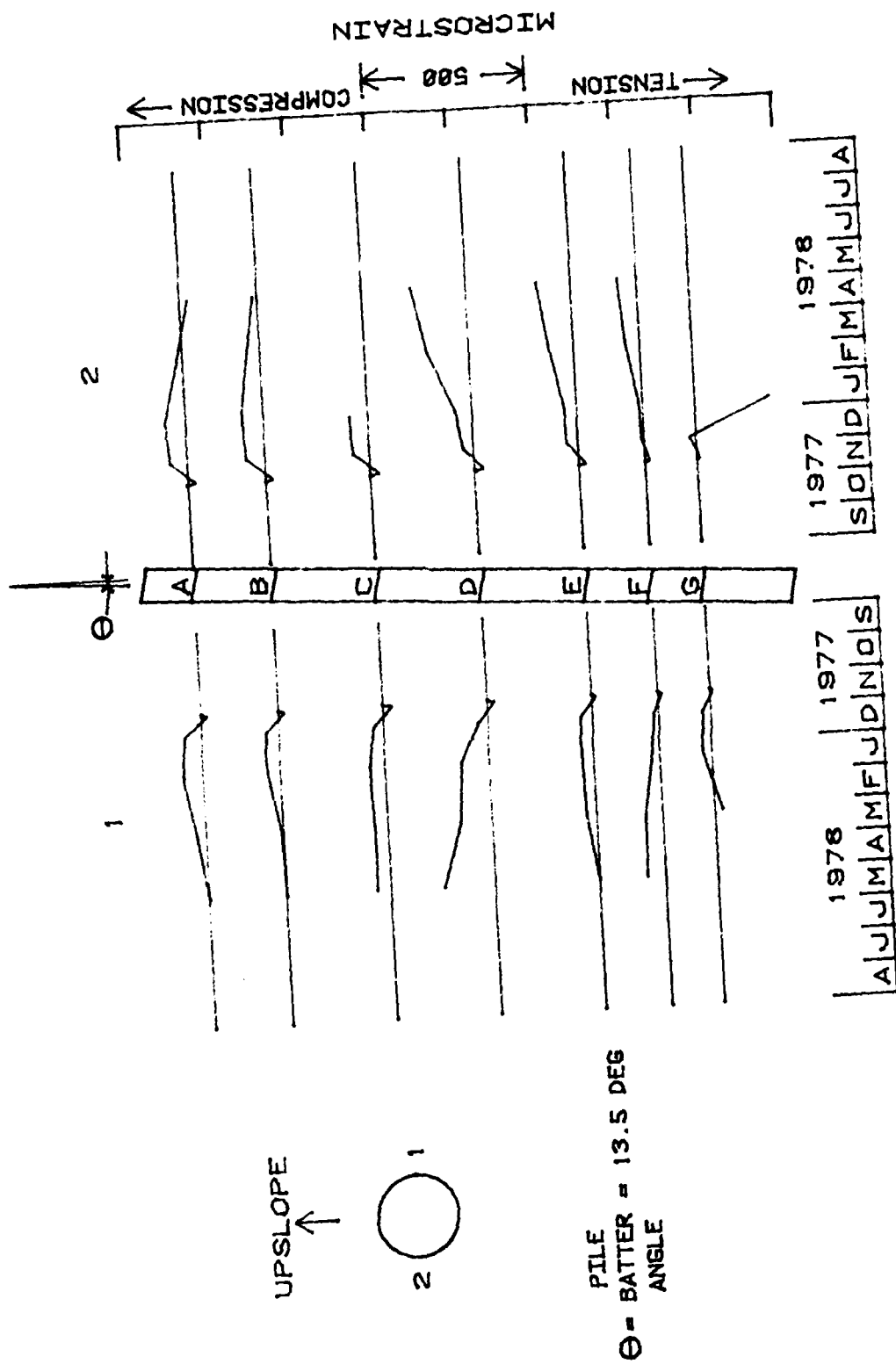


Figure 64. Strain-time histories along the length of pile 404, upstation and downstation

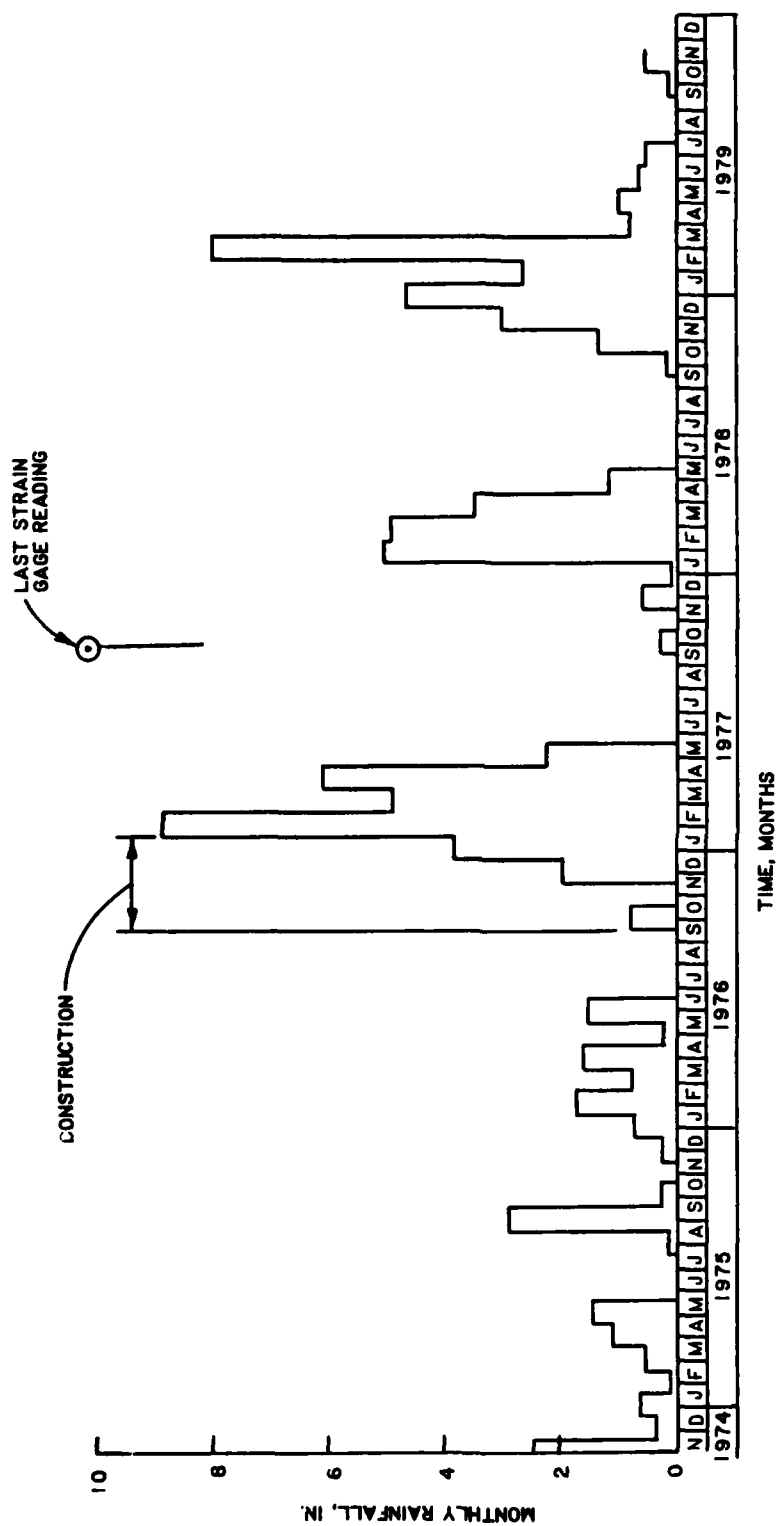
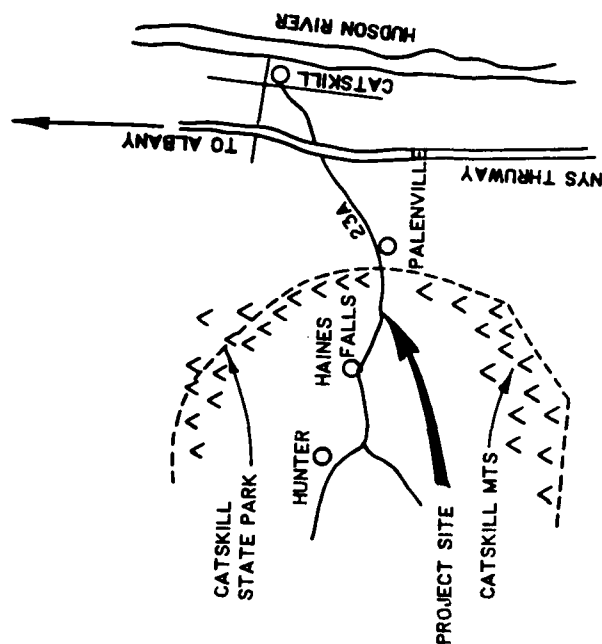
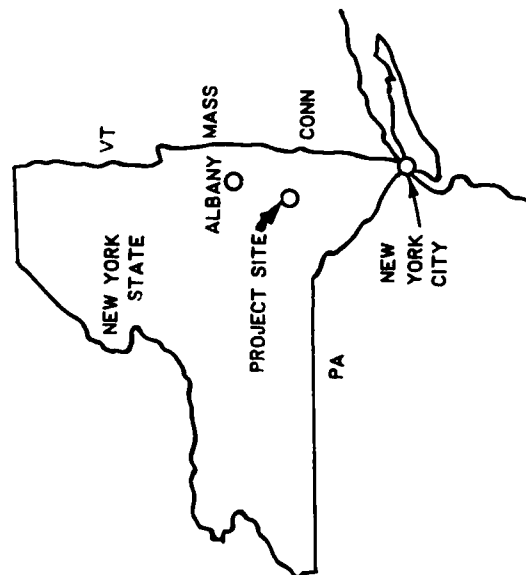


Figure 65. Rainfall data near project site



B. DETAILED LOCATION



A. GENERAL LOCATION

Figure 66. Location map for New York site on Route 23A near Palenville, N. Y.

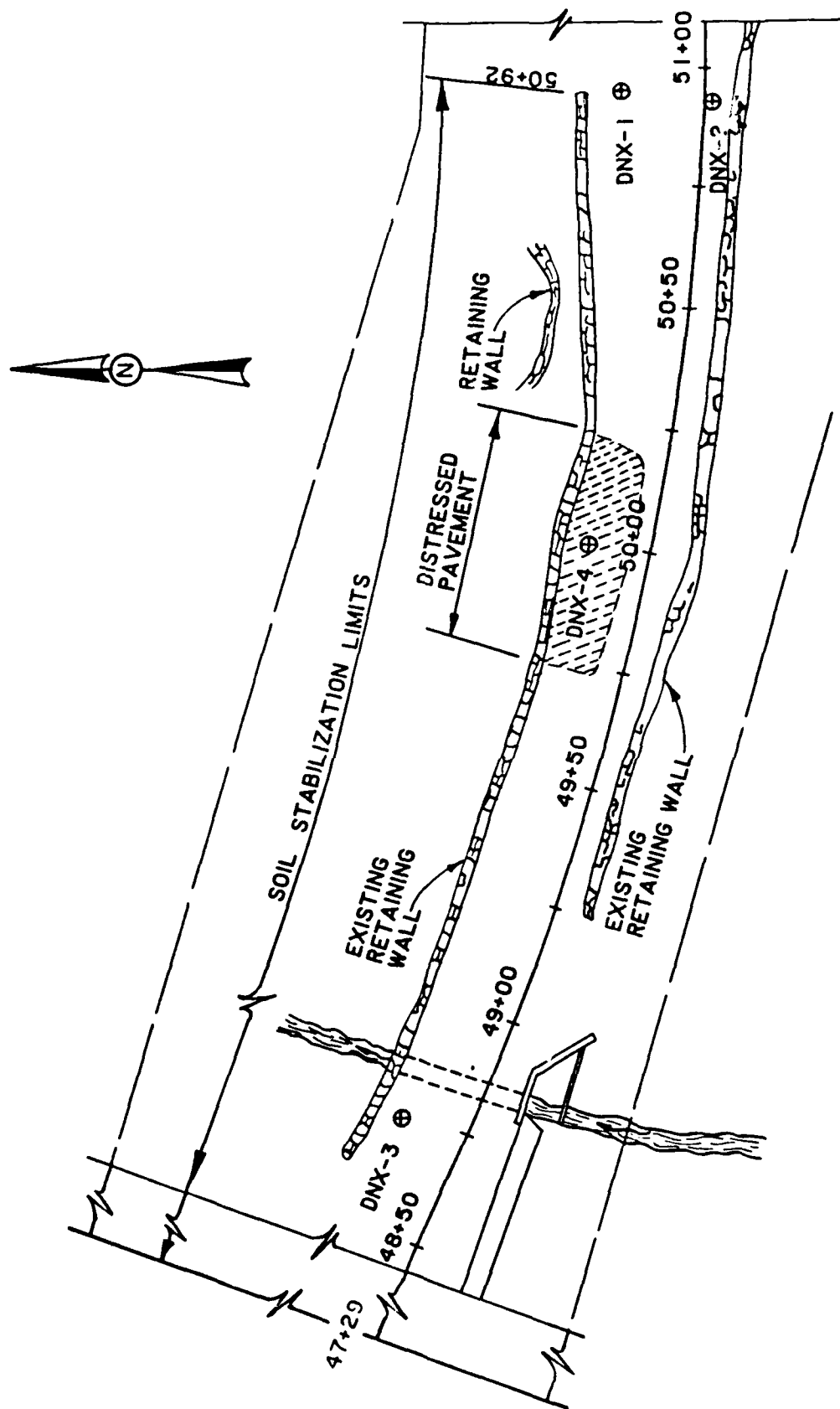


Figure 67. Plan view of road project



d.



b.



c.

Figure 68. Selected views of road conditions along
Route 23A

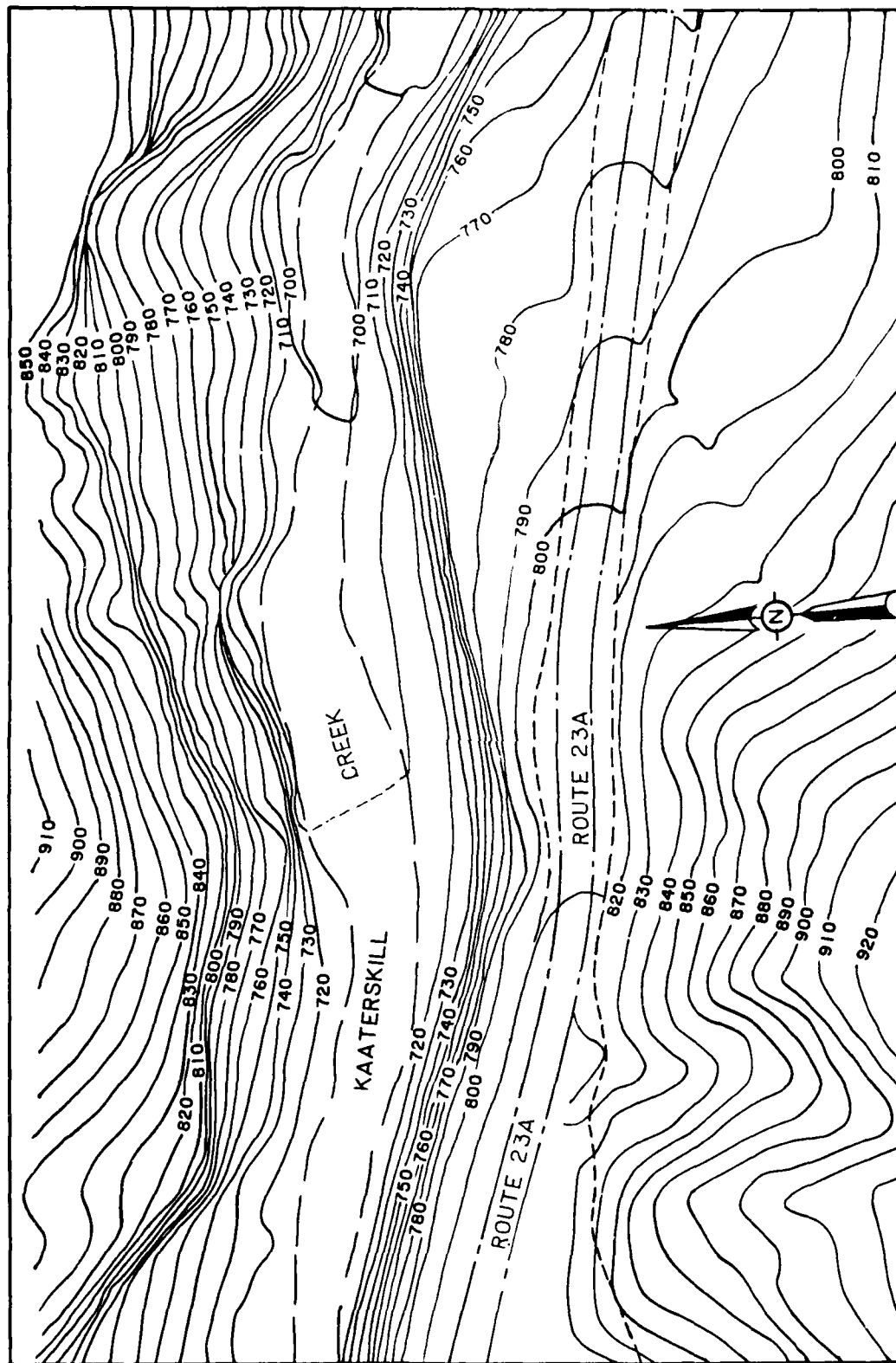


Figure 69. Surface contours at project

NOTE: ONLY THOSE DRILL HOLES WHICH ENCOUNTERED BEDROCK ARE SHOWN.

STEEP ROCK FACE 70± FT TO STREAM

ROUTE 23A

EXISTING BOX CULVERT

EXISTING METAL BIN WALL

APPROX. POSITION AND EXTENT OF RETICULATED RETAINING STRUCTURE

PLAN

LEGEND

⊕	ON DRILL HOLE
—	SI SLOPE INDICATOR HOLE
---	SOLID CONTOURS INTERPOLATED
----	DASHED CONTOURS INFERRED
-.-.-	DASH/DOT CONTOURS IN EXPOSED BEDROCK AREAS

Figure 70. Top of rock contours at project

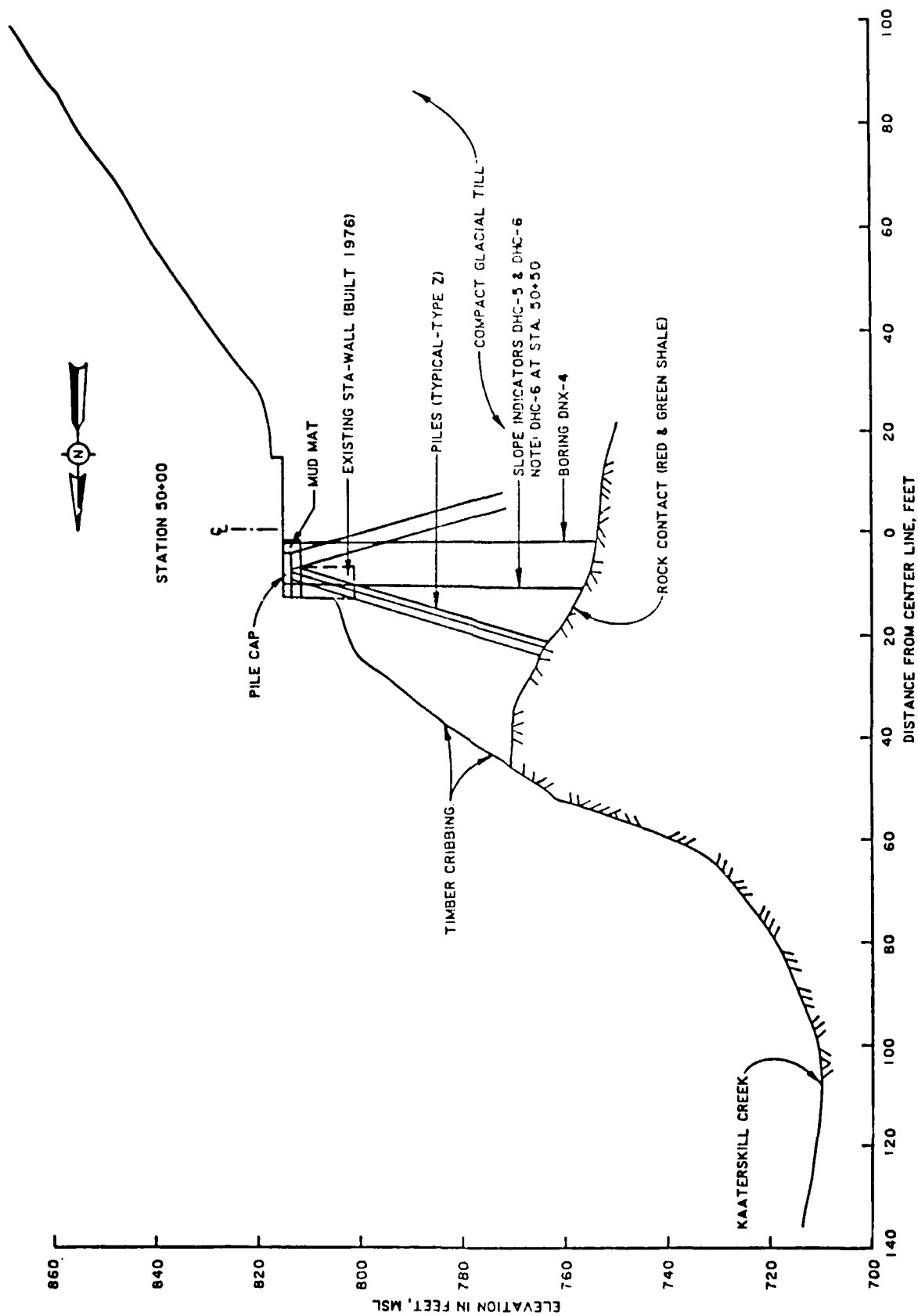
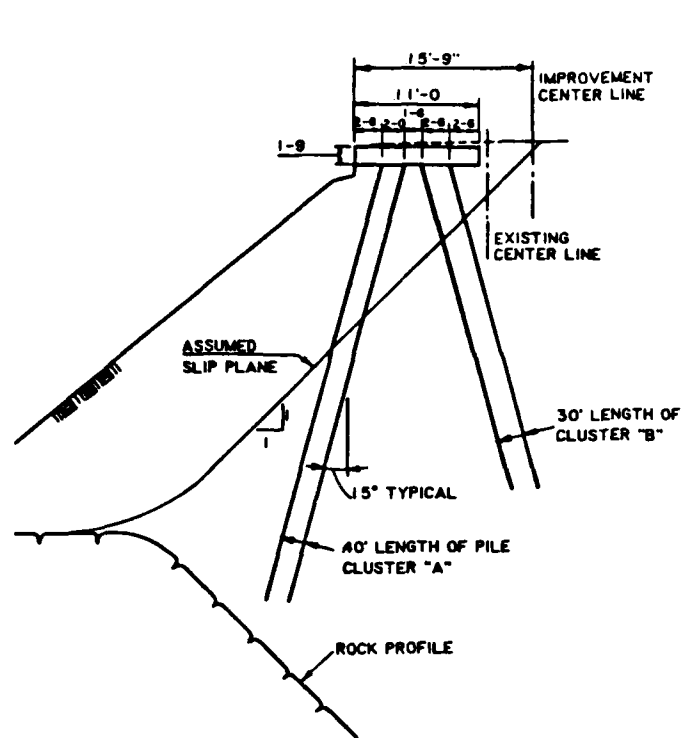
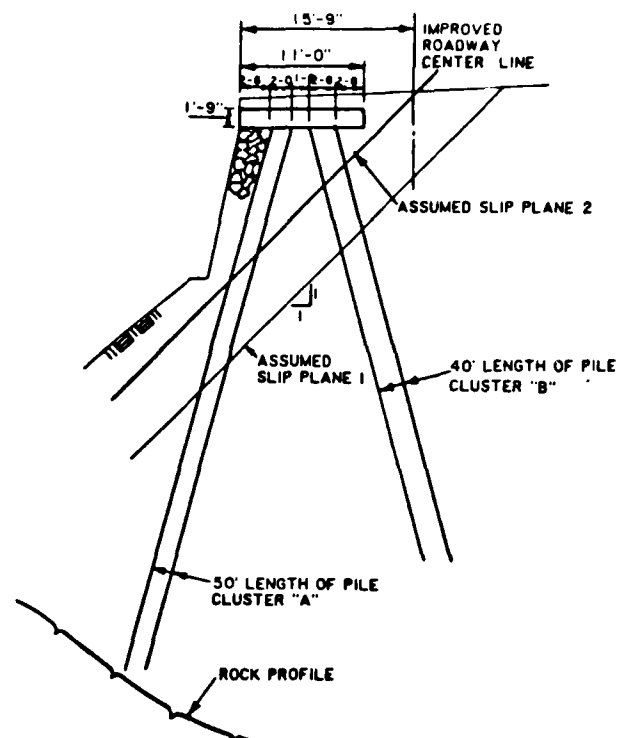


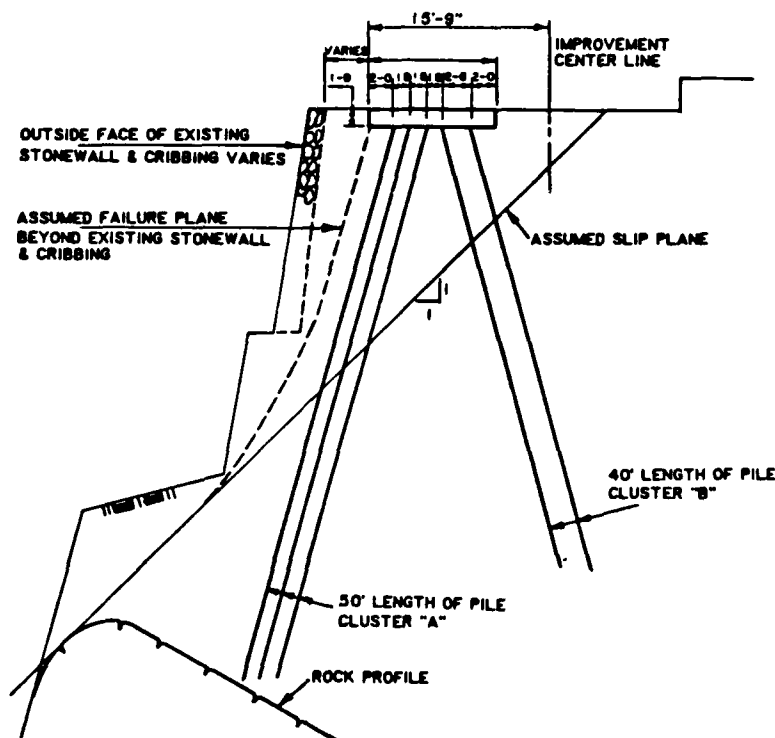
Figure 71. Profile at sta 50+00



SECTION 'X' LOOKING EAST

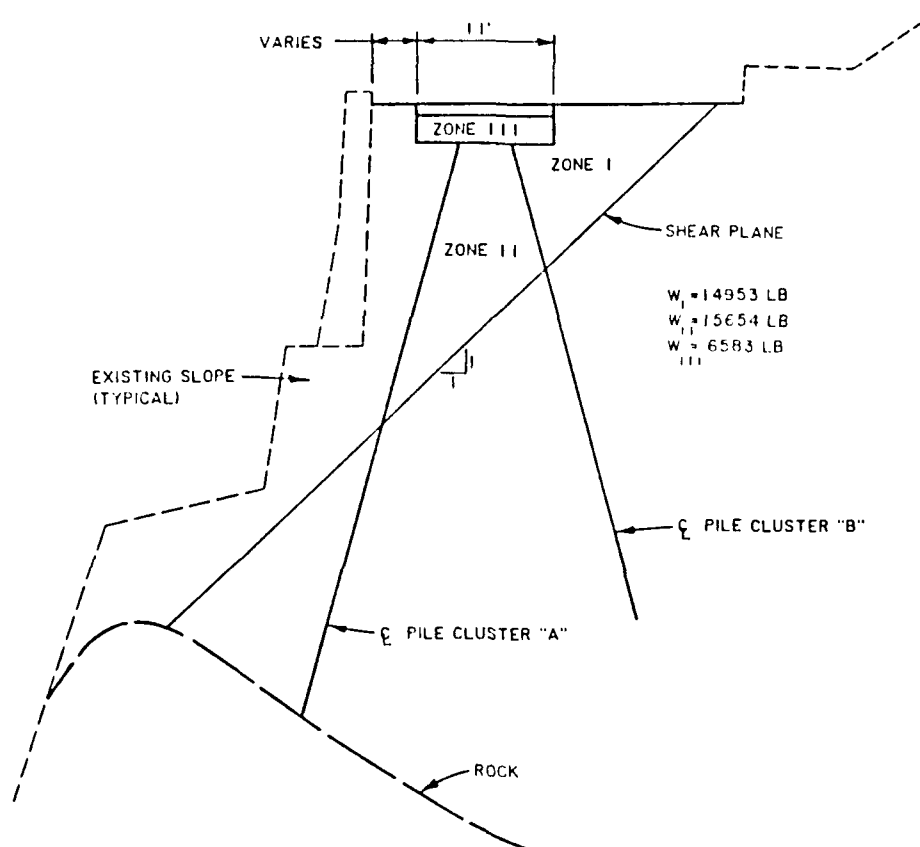


SECTION 'Y' LOOKING EAST



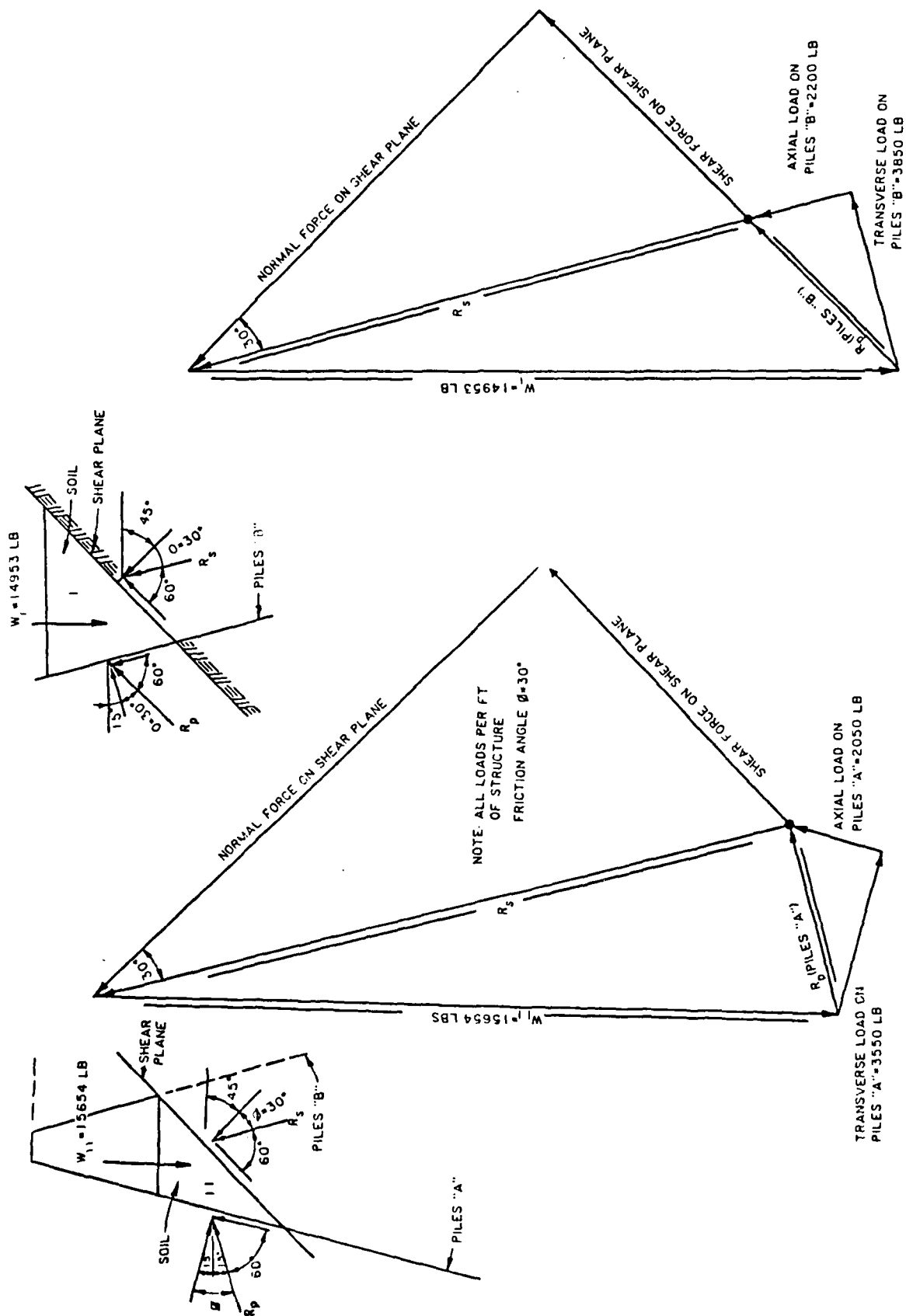
SECTION 'Z' LOOKING EAST

Figure 72. Pile types X, Y, and Z



a. SECTION Z-Z

Figure 73. Design approach - type Z piles (Sheet 1 of 3)



b. GRAPHICAL SOLUTION-SOIL WEDGE FORCES

TRANSVERSE LOAD ON CLUSTER "A"=3550

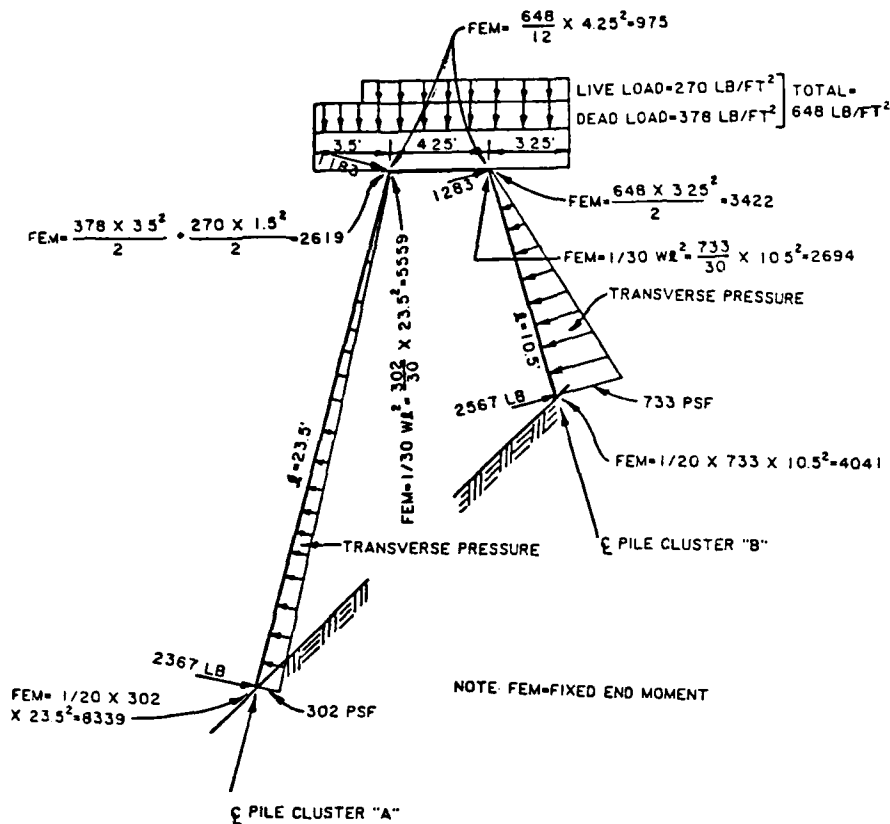
TRIANGULAR END PRESSURE

$$= \frac{3550 \times 2}{235} = 302 \text{ PSF}$$

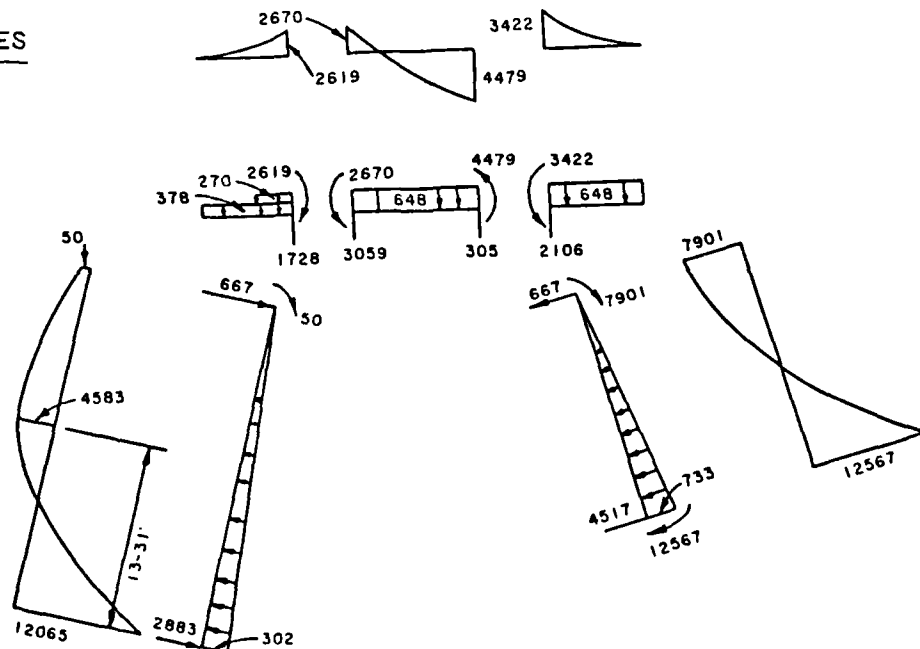
TRANSVERSE LOAD ON CLUSTER "B"=3850

TRIANGULAR END PRESSURE

$$= \frac{3850 \times 2}{105} = 733 \text{ PSF}$$



C. FORCES ON PILES



d. COMBINED MOMENTS AND SHEARS

Figure 73. (Sheet 3 of 3)

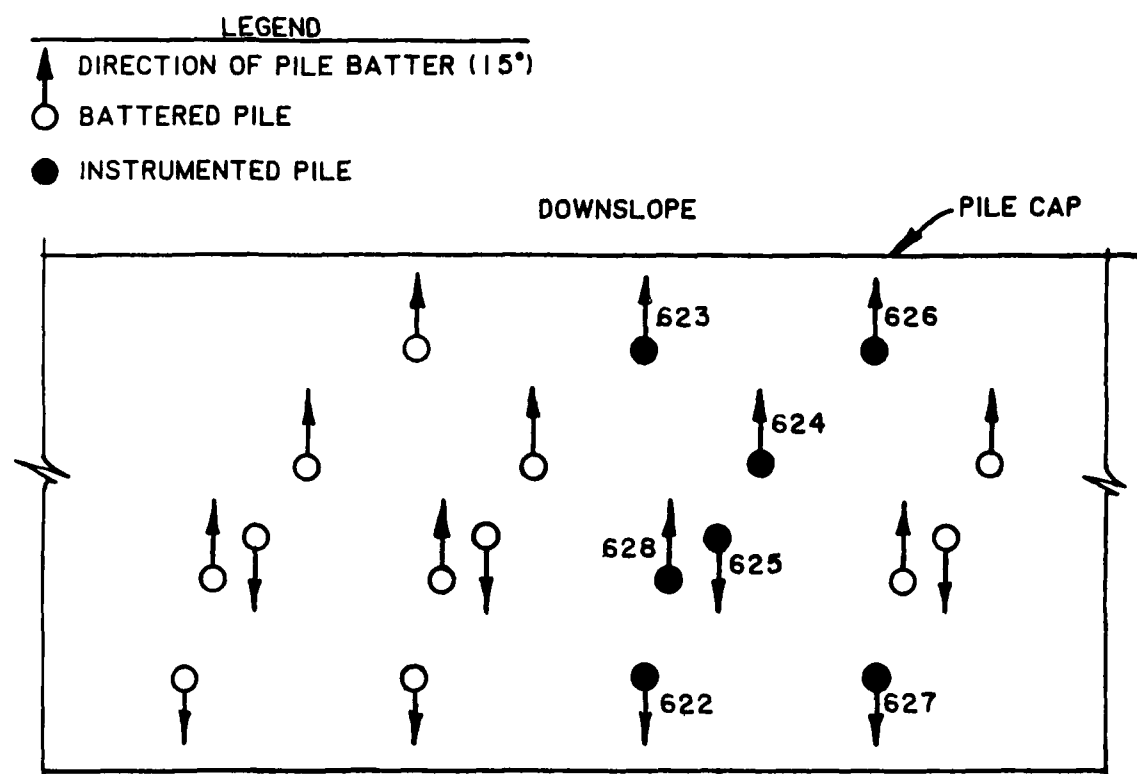


Figure 74. Plan view of pile layout

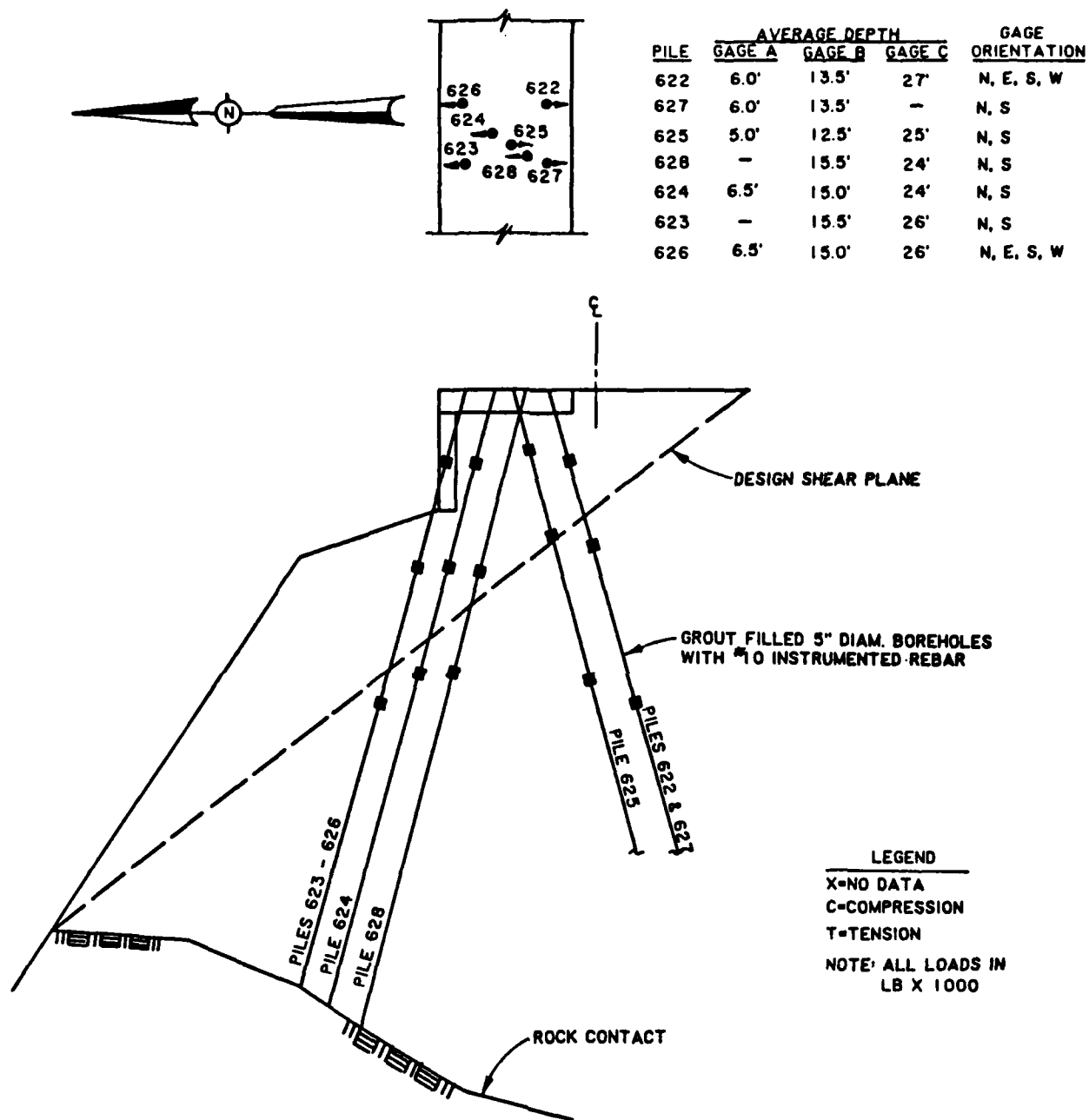


Figure 75. Gage depths of instrumented piles



a. Looking east toward sta 50+00



b. Looking east toward sta 51+00

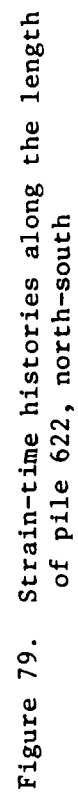
Figure 76. Views of construction



Figure 77. View showing layout of instrumented piles



Figure 78. View at area of instrumented piles showing mud mat elevation relative to existing pavement



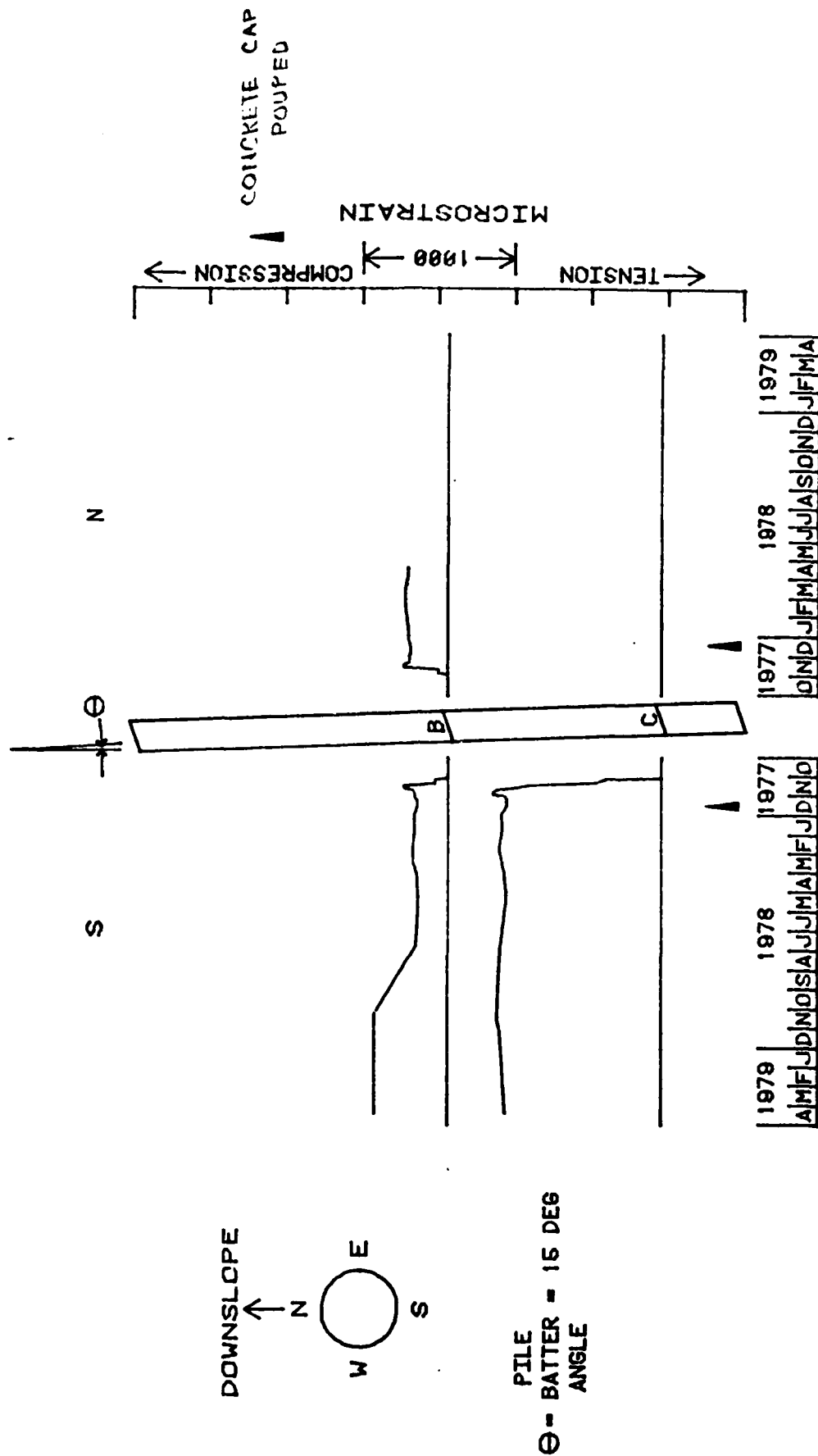


Figure 81. Strain-time histories along the length of pile 623

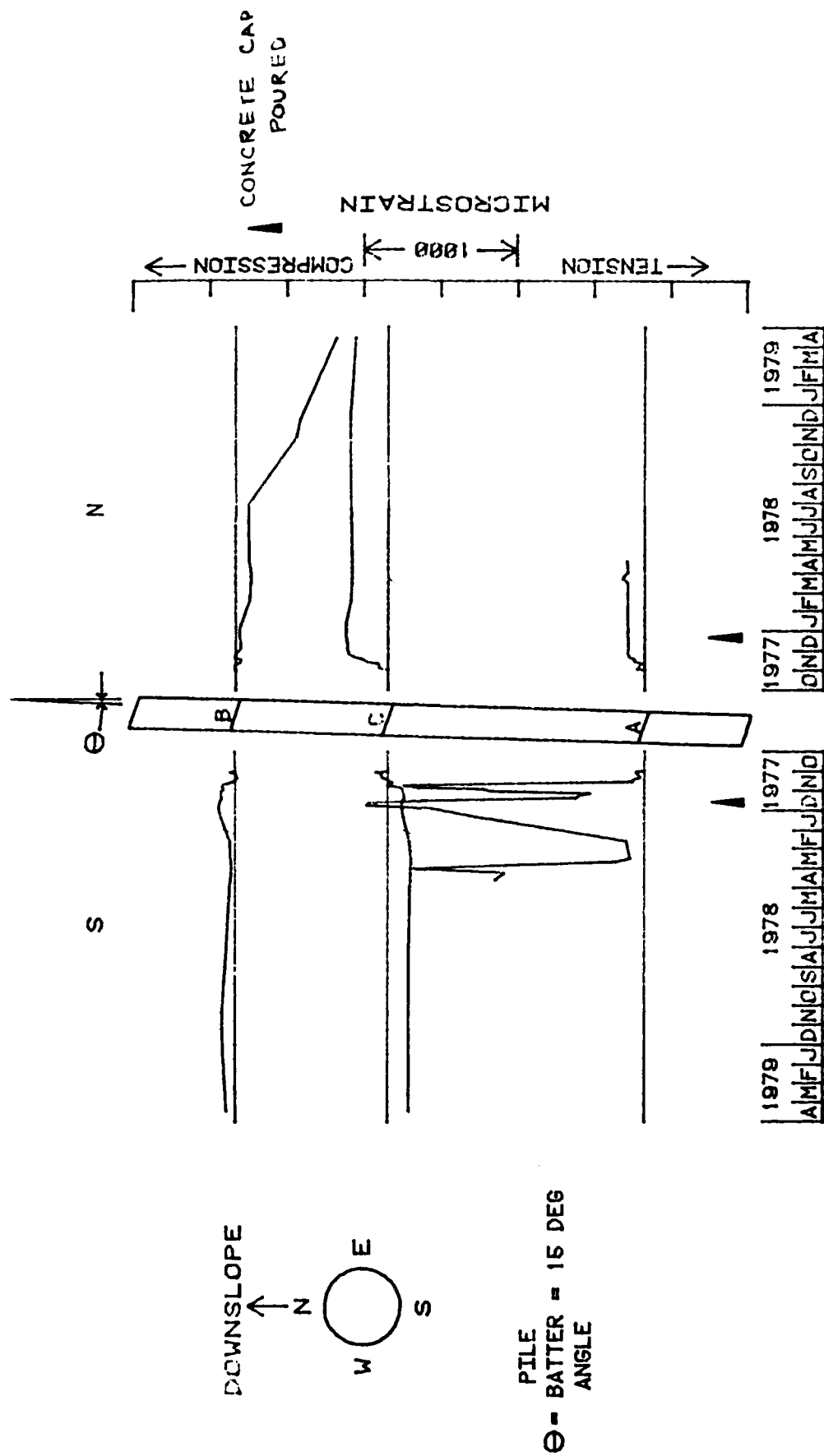


Figure 83. Strain-time histories along the length of pile 625

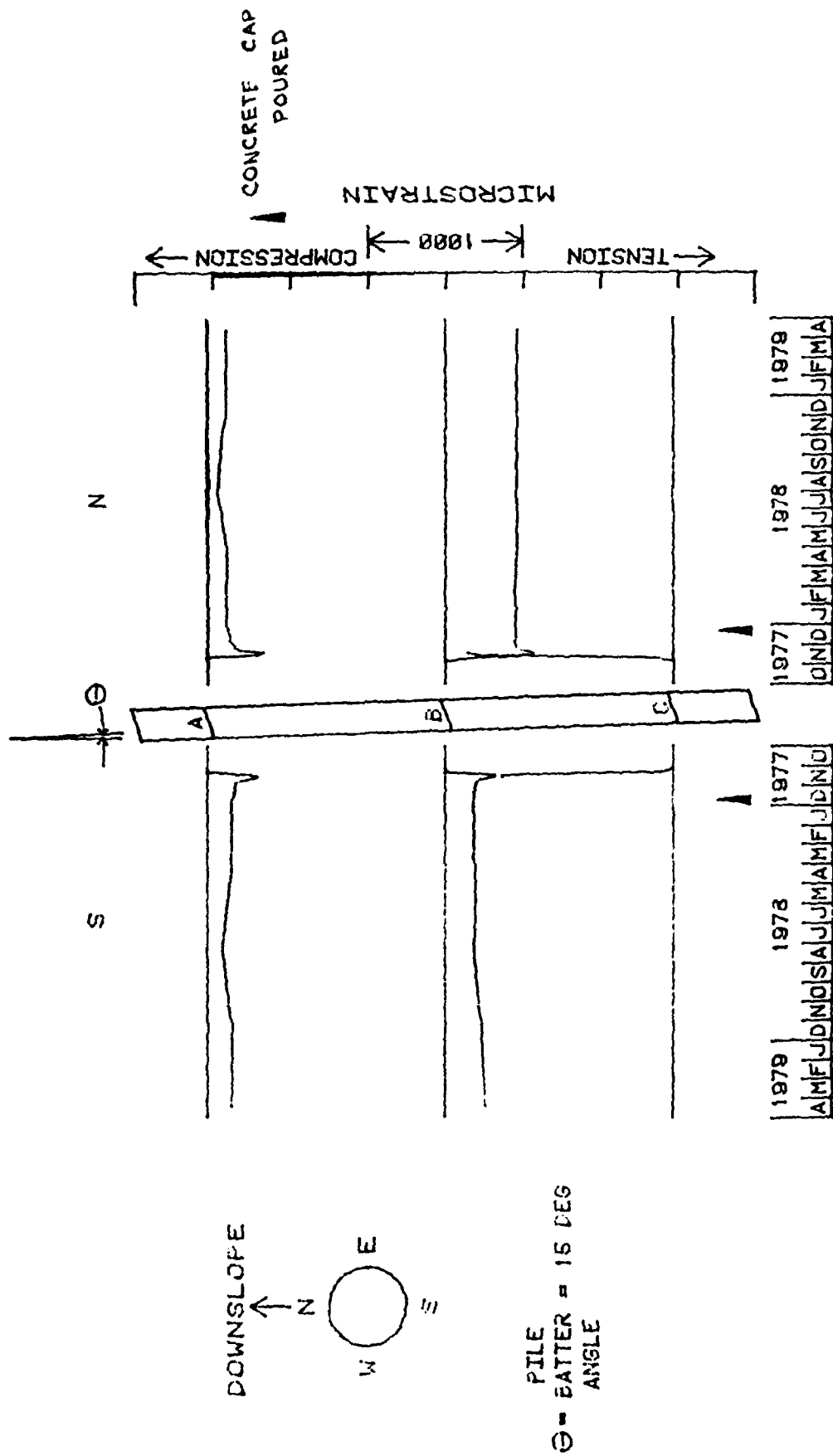


Figure 84. Strain-time histories along the length of pile 626, north-south

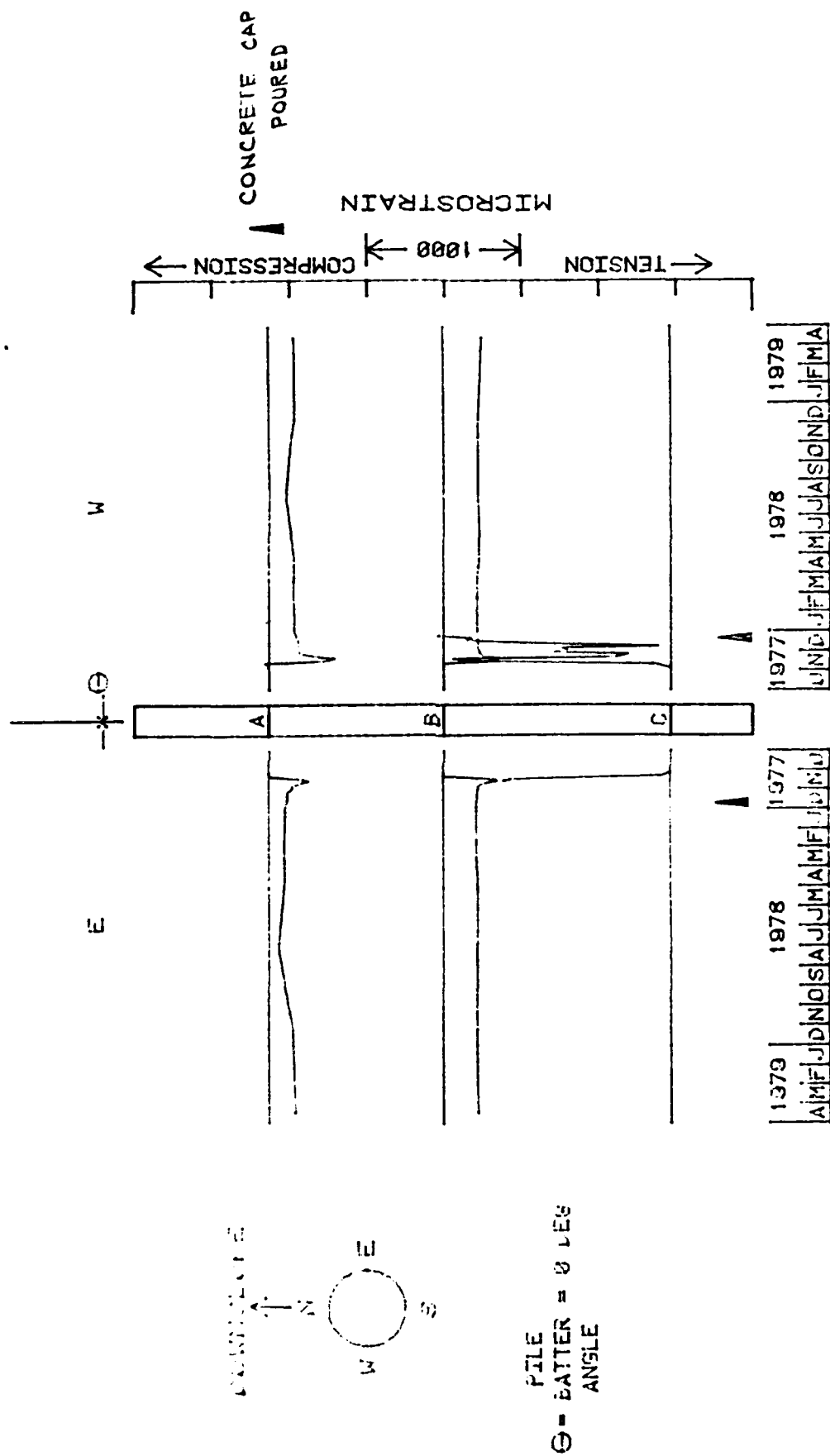


Figure 85. Strain-time histories along the length of pile 626, east-west

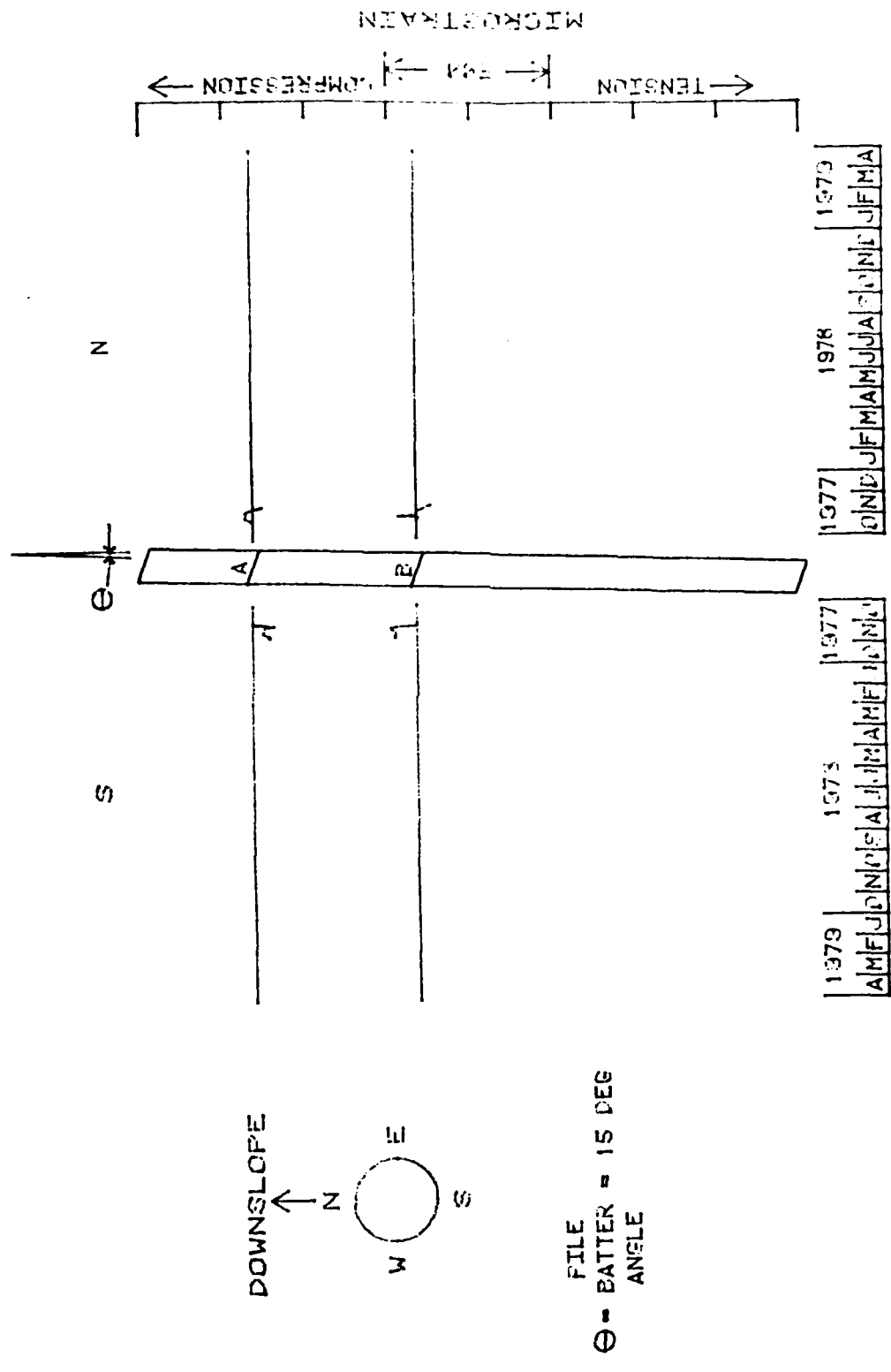


Figure 86. Strain-time histories along the length of pile 627

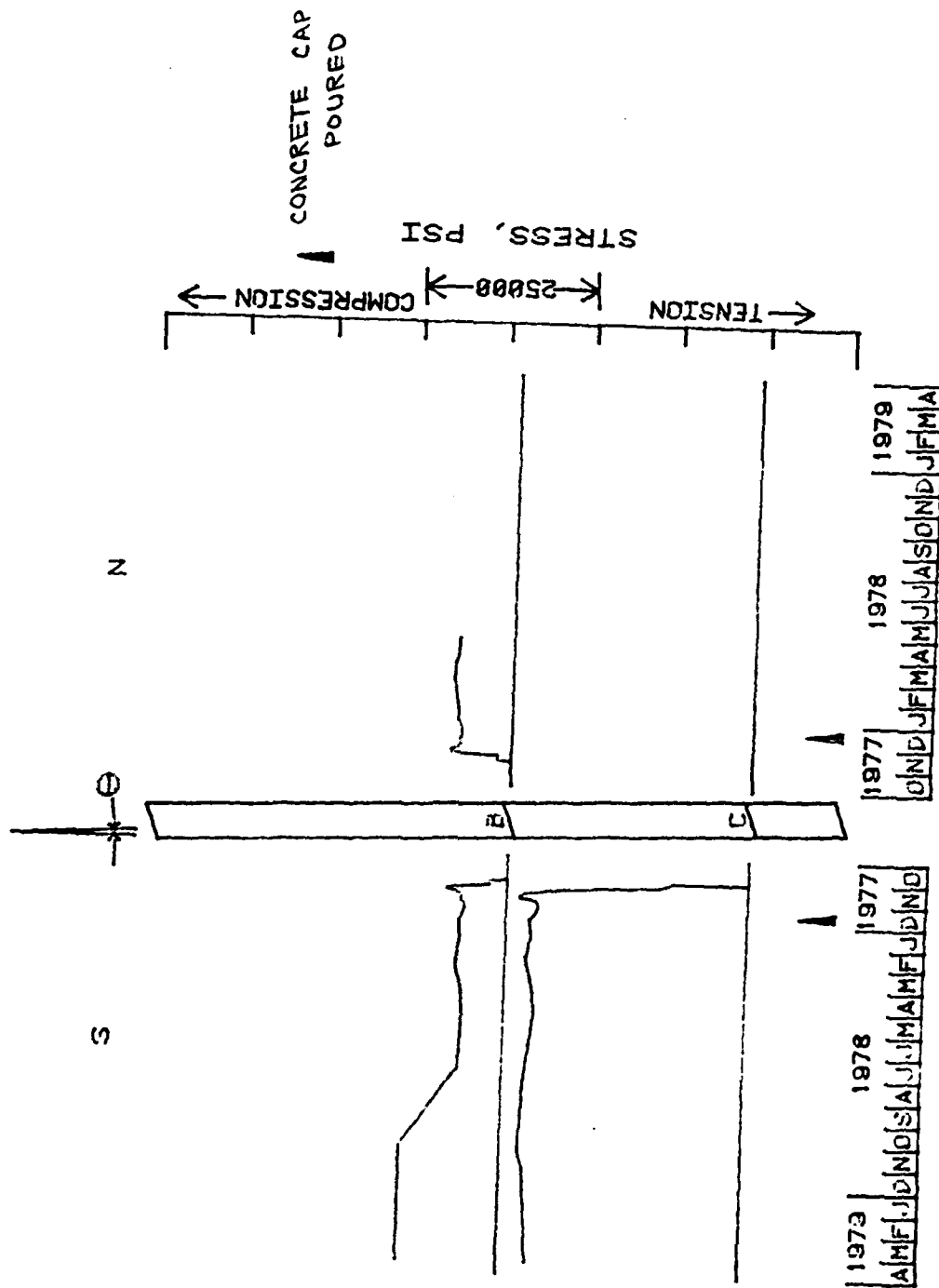


Figure 90. Stress-time histories along the length of pile 623

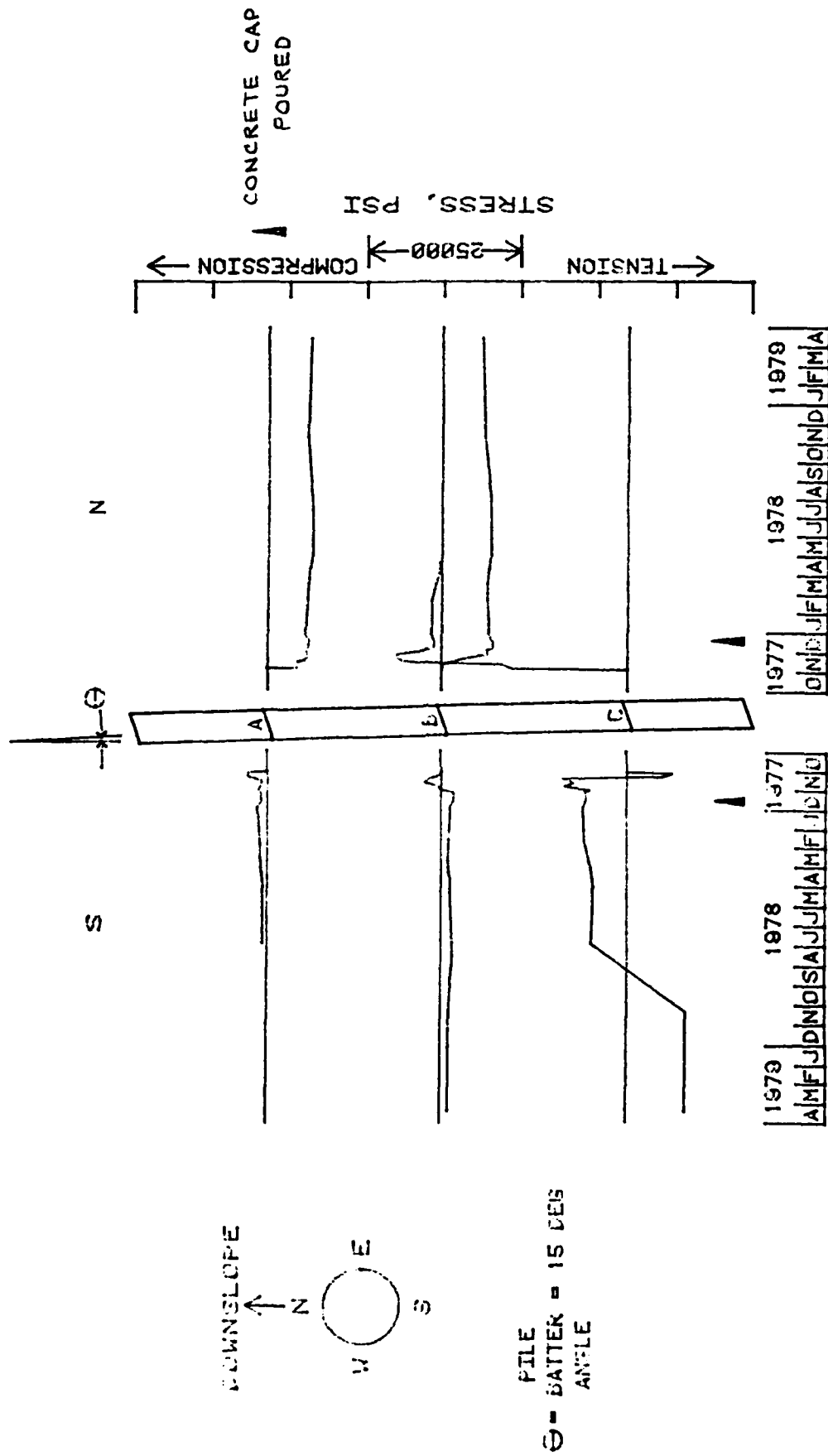


Figure 91. Stress-time histories along the length of pile 624

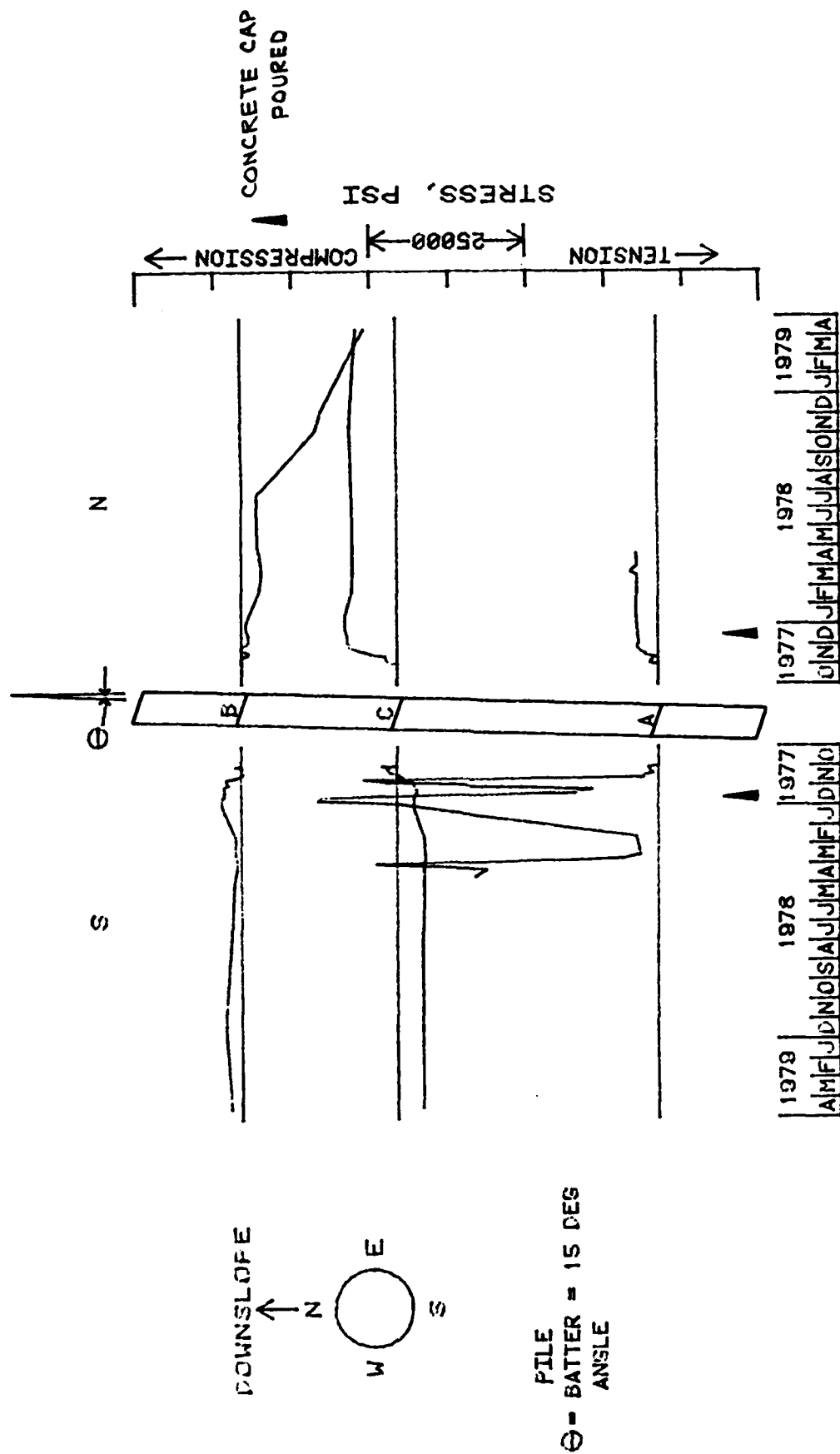


Figure 92. Stress-time histories along the length of pile 625

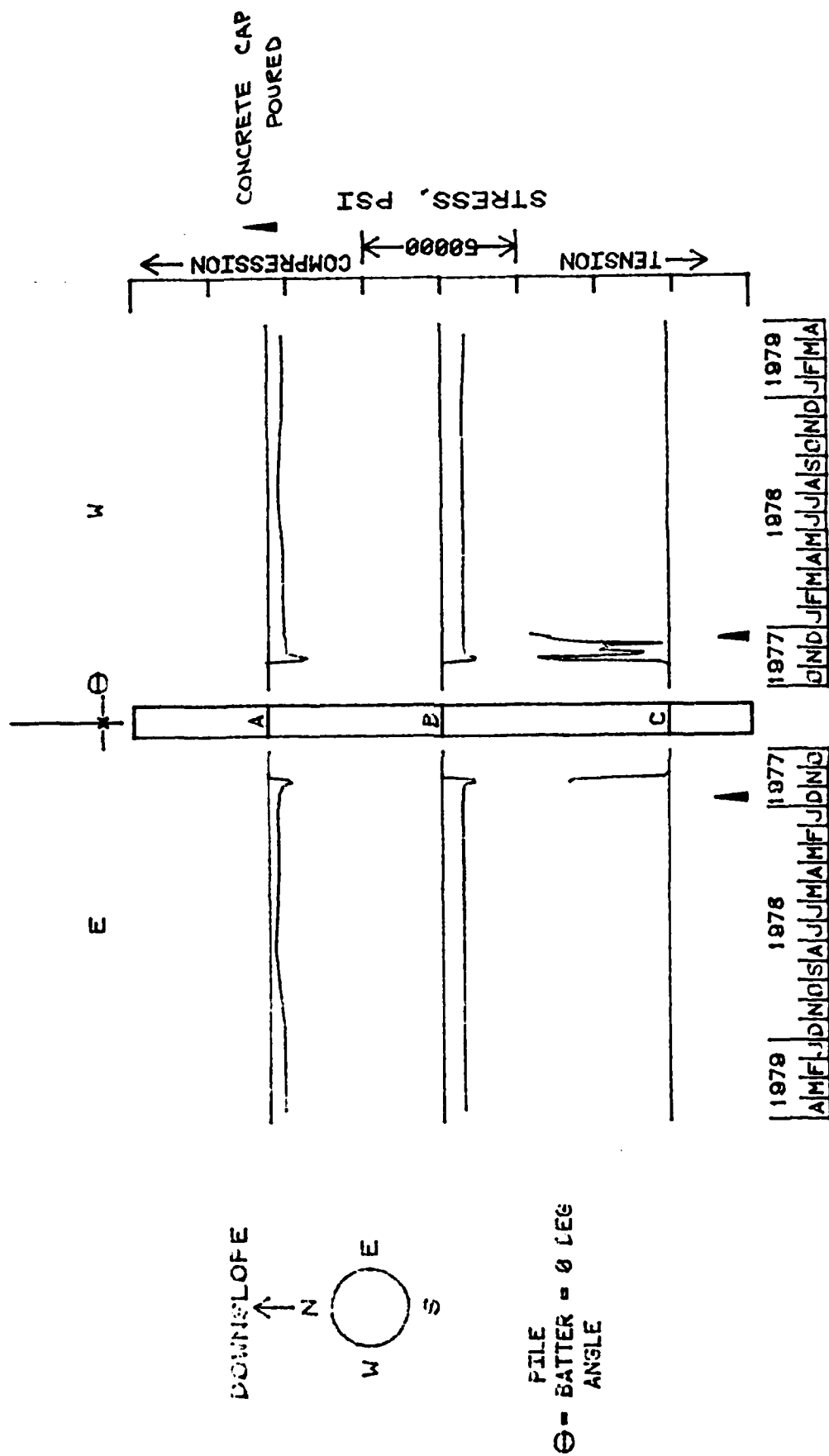
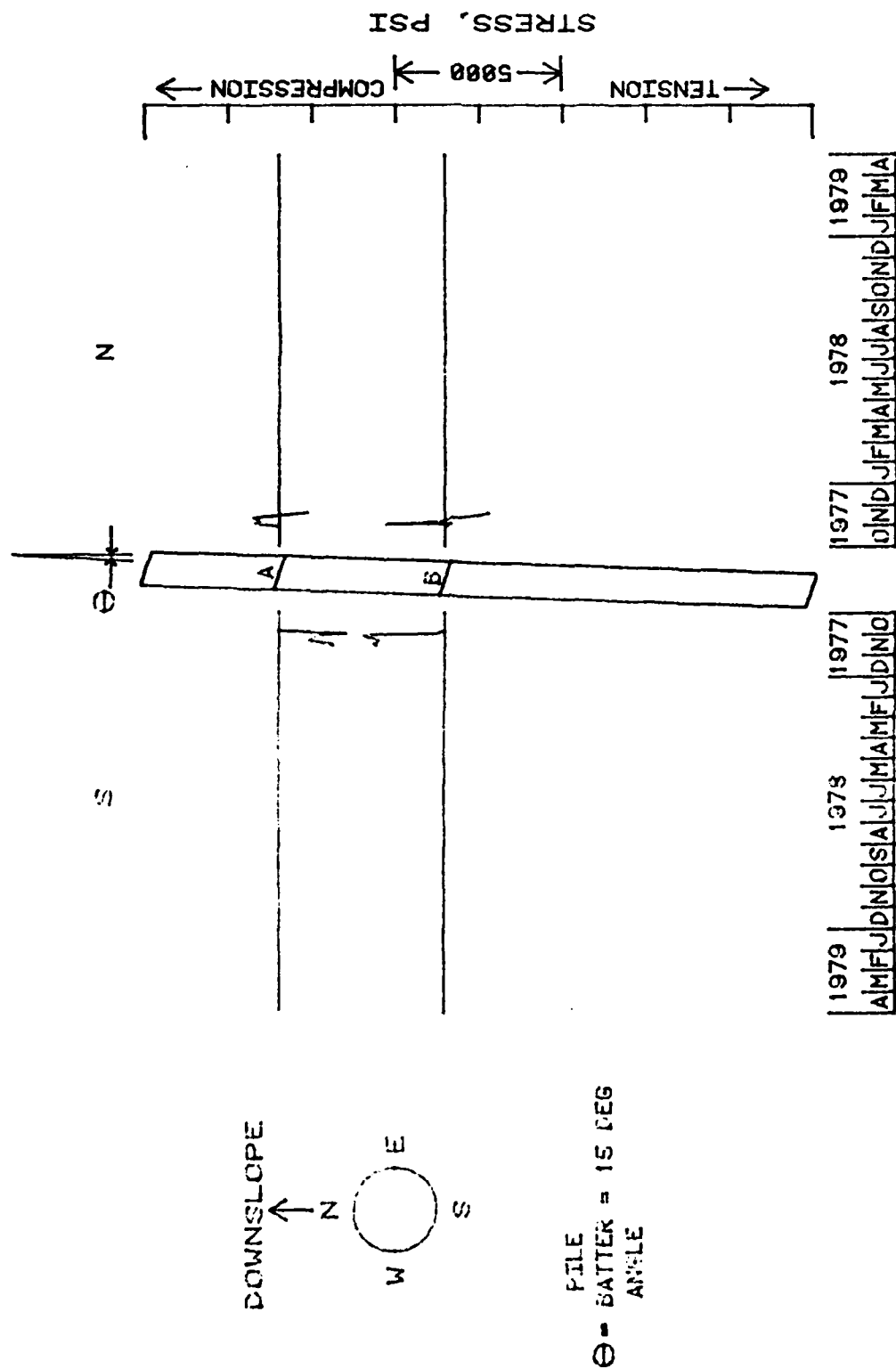


Figure 94. Stress-time histories along the length of pile 626, east-west



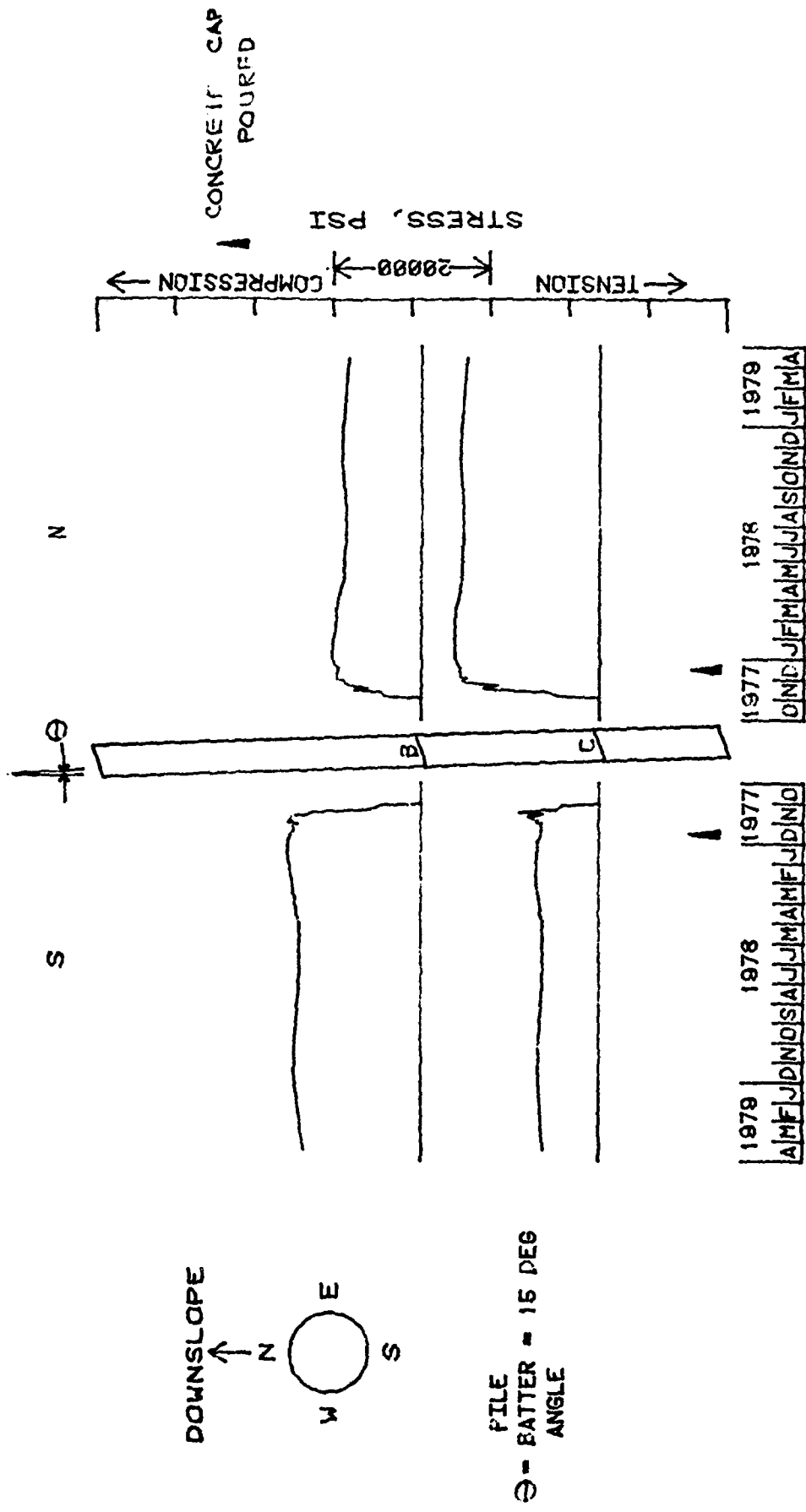
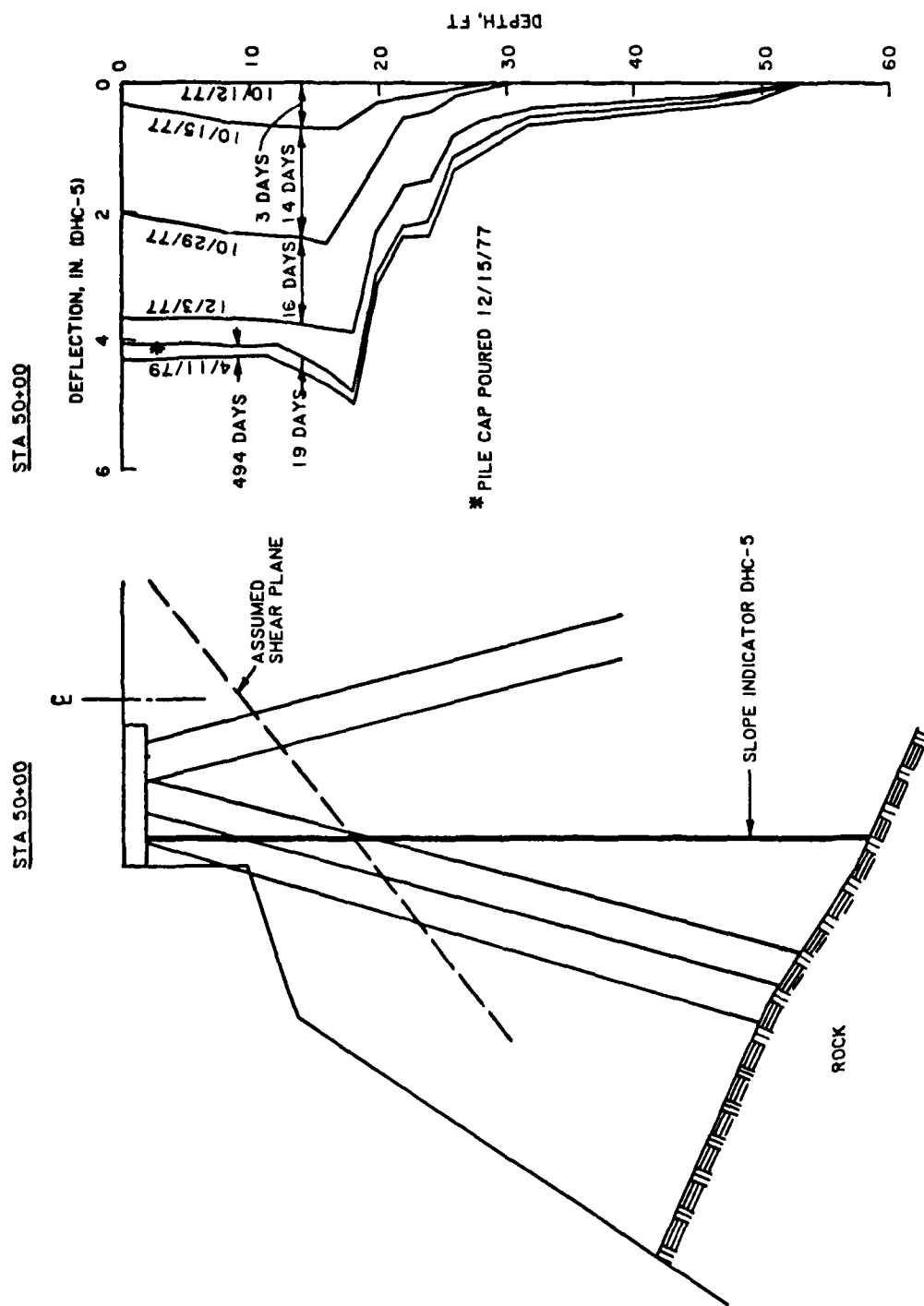
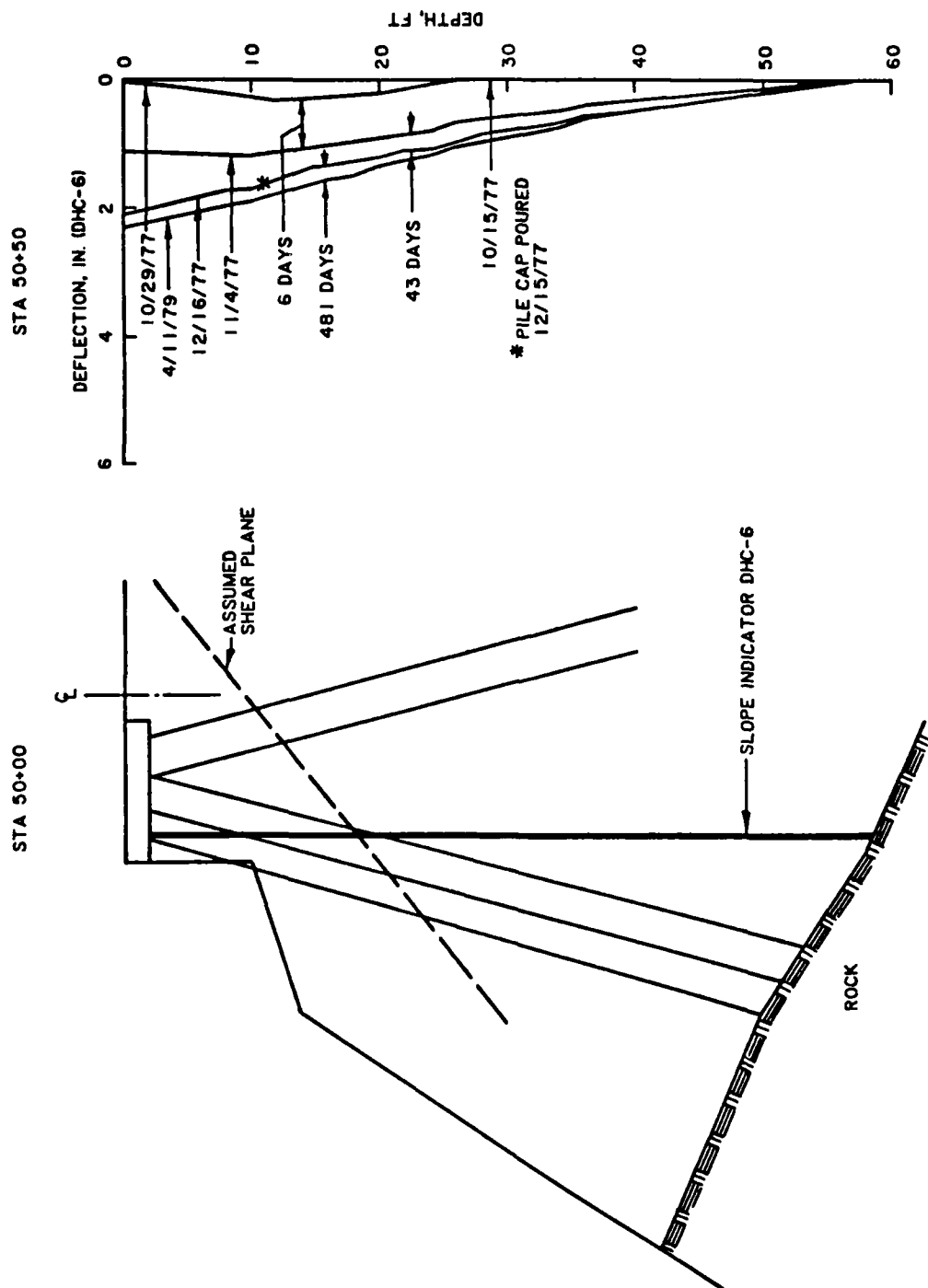


Figure 96. Stress-time histories along the length of pile 628



2. STATION 50+00, DHC-5

Figure 97. Slope indicator movements (Continued)



b. STATION 50+50, DHC-6

Figure 97. (Concluded)

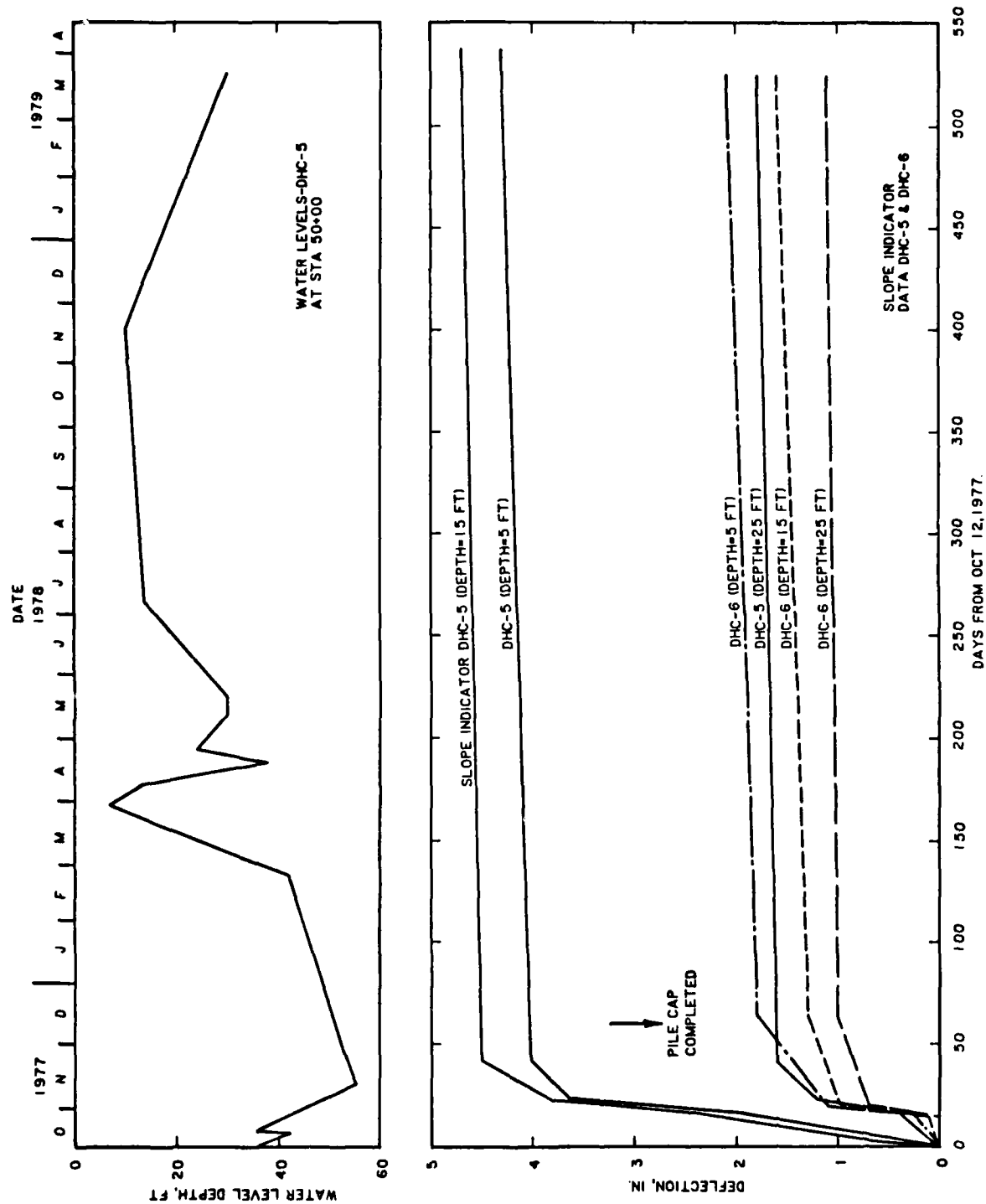


Figure 98. Slope indicator deflections and water level readings

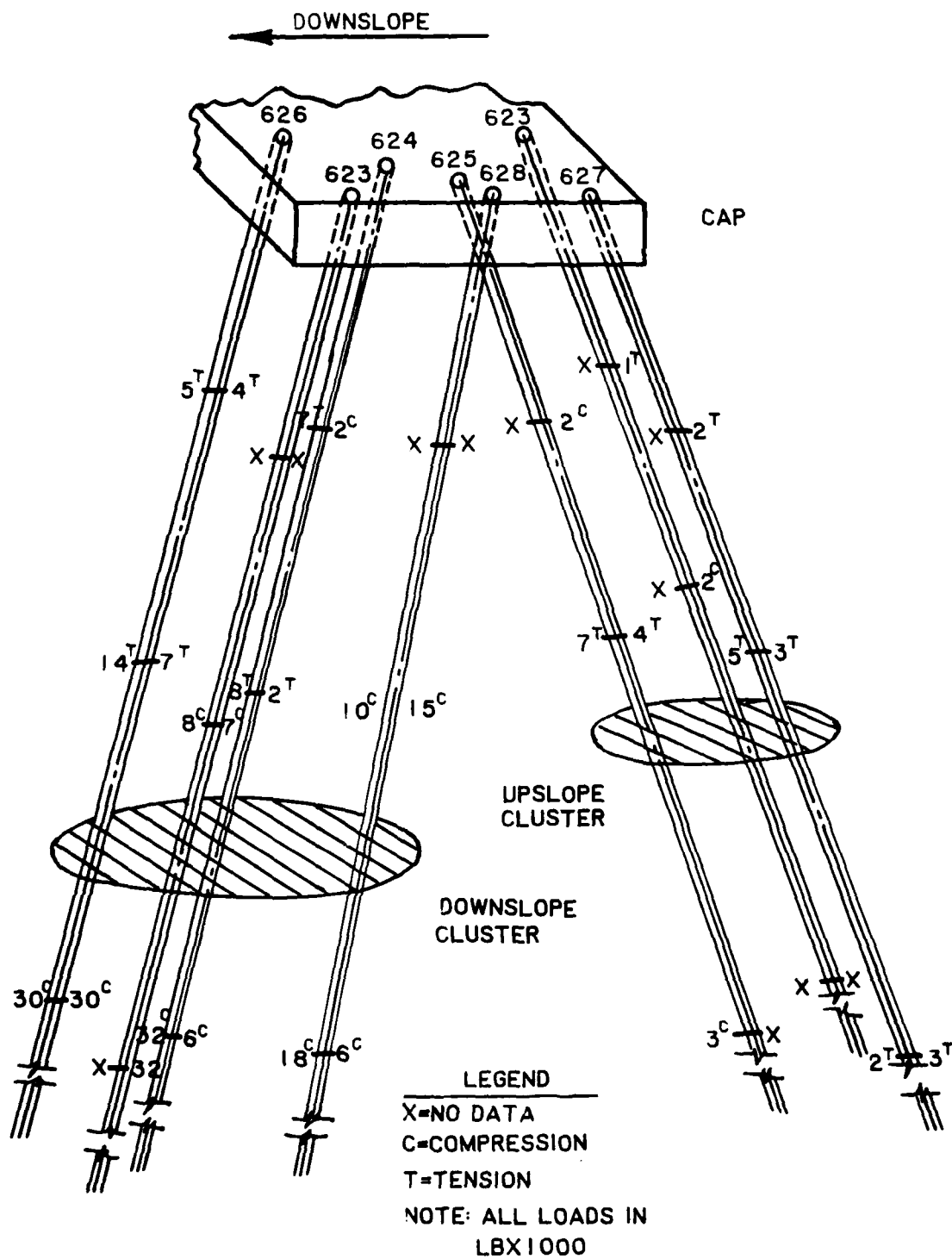


Figure 99. Strain gage loads

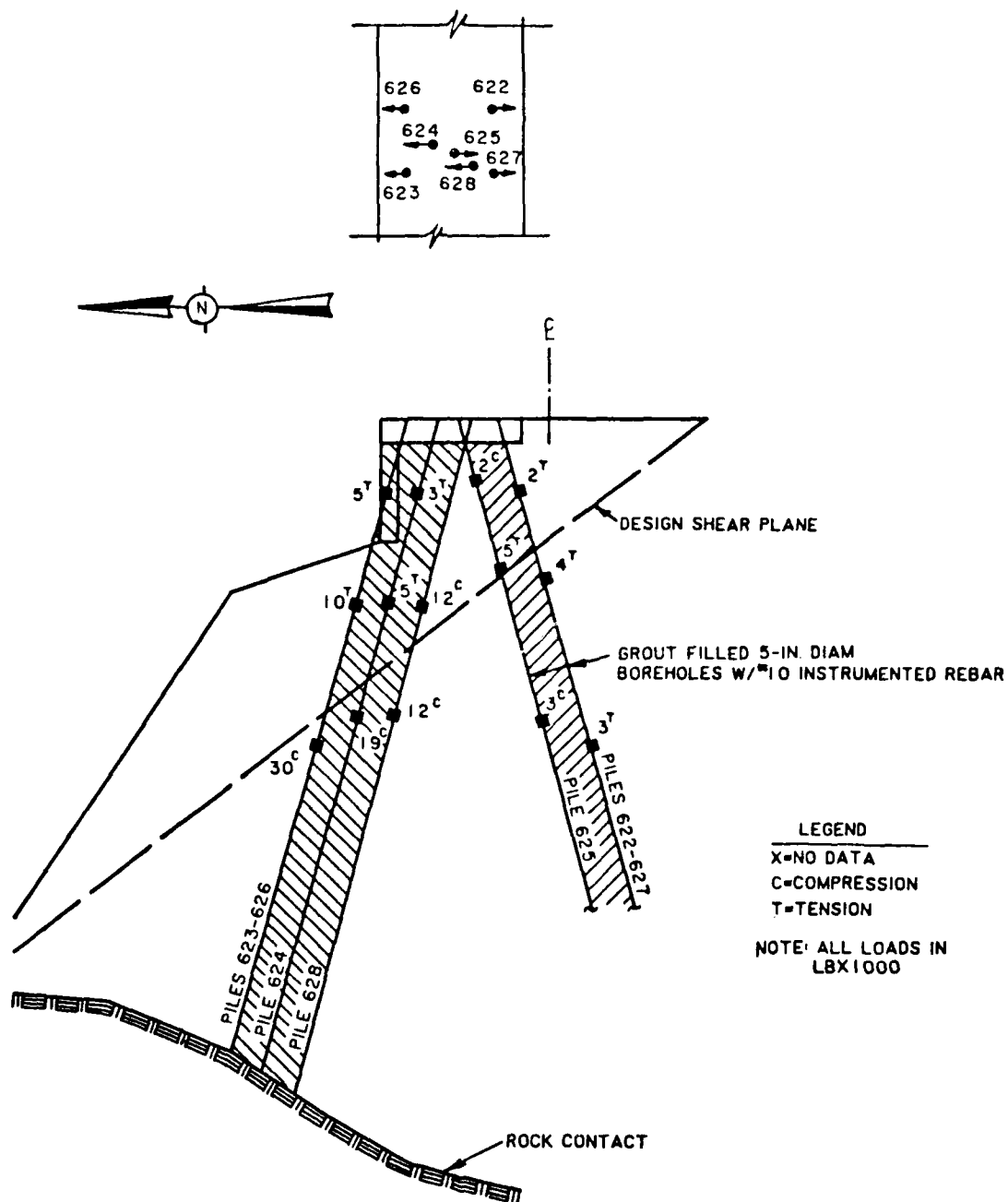


Figure 100. Average combined loads in "beams"

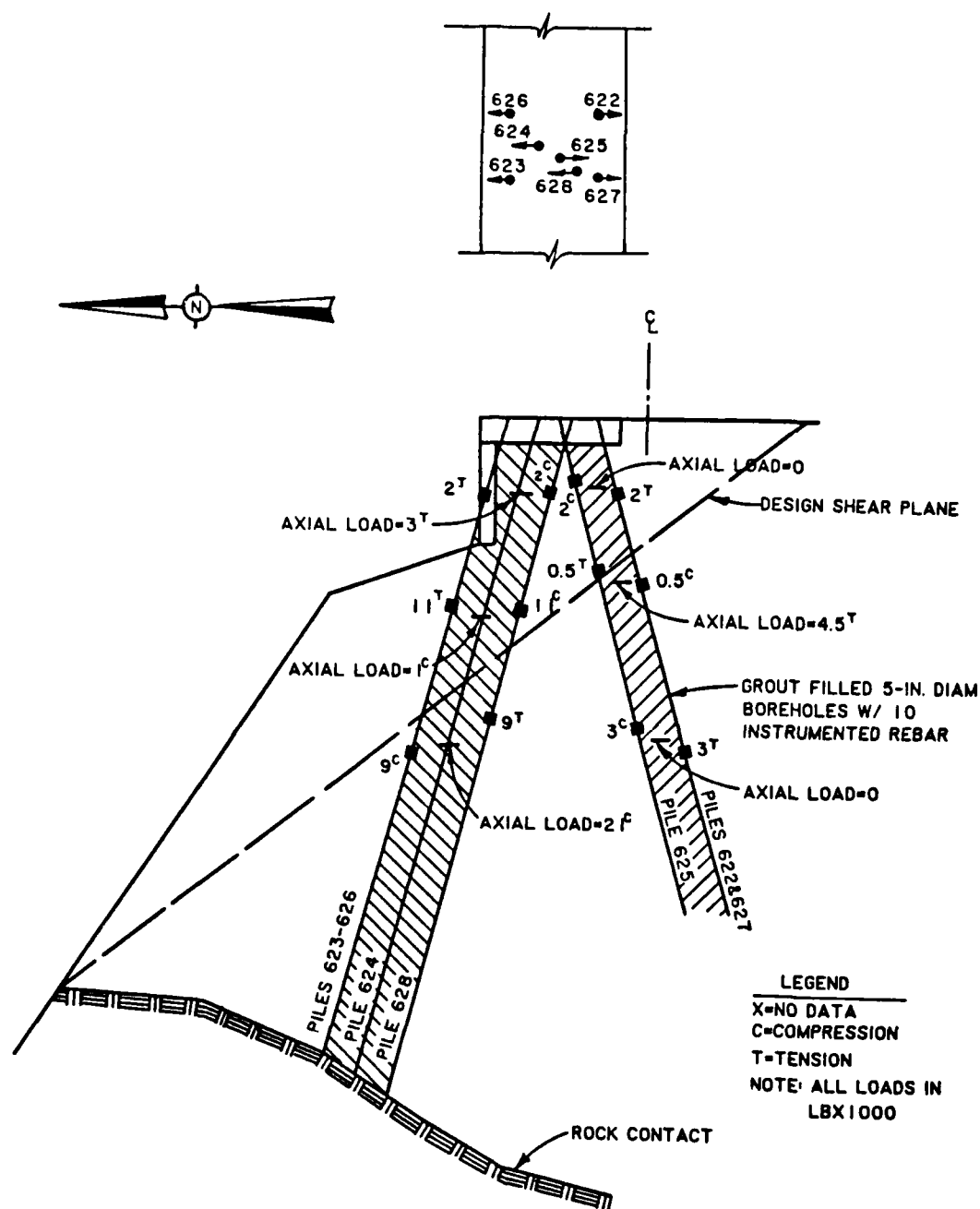


Figure 101. Interpreted bending and axial loads in "beams"

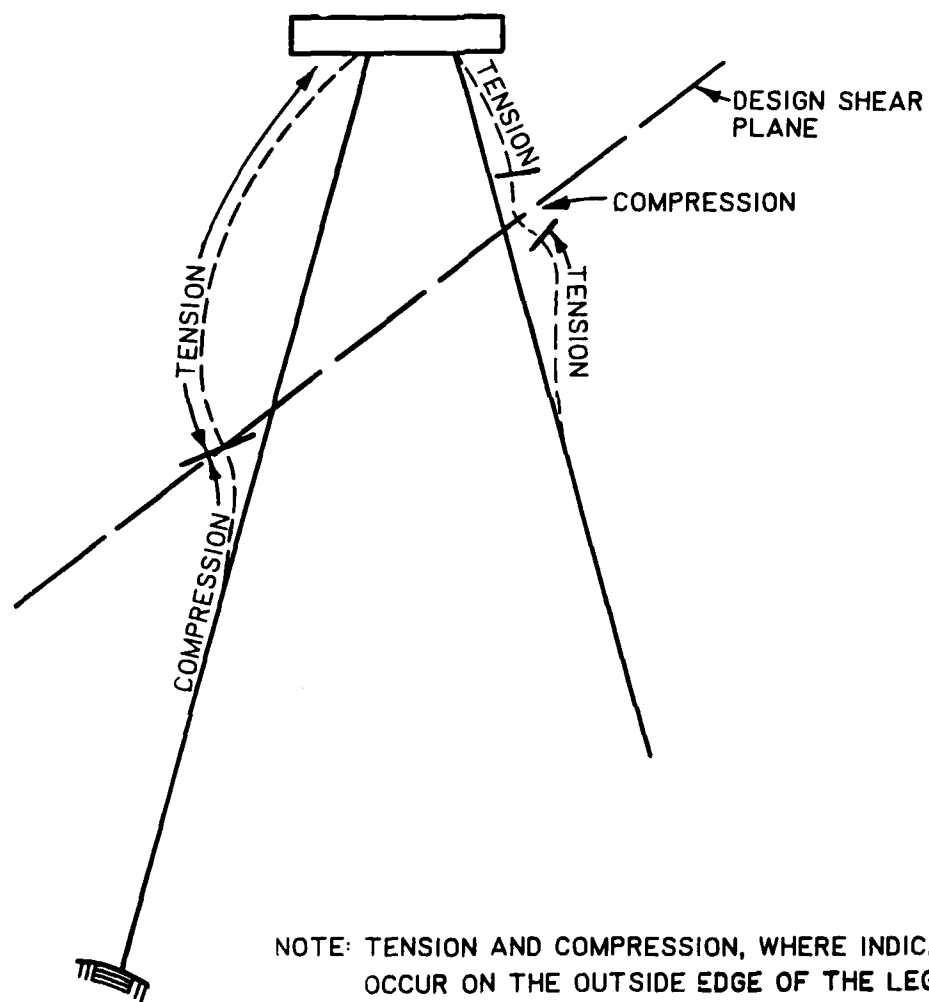


Figure 102. Interpreted bending of "beams"

END

FILMED

8